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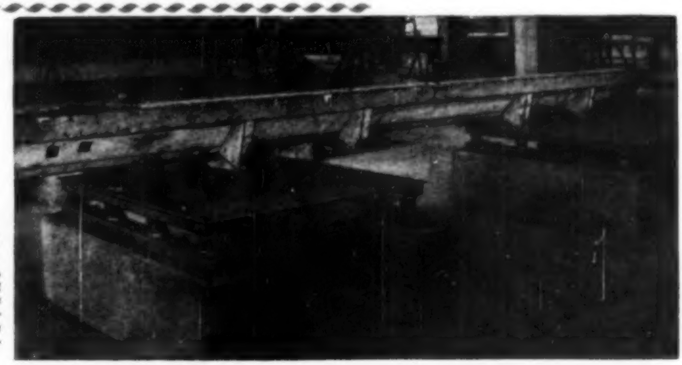

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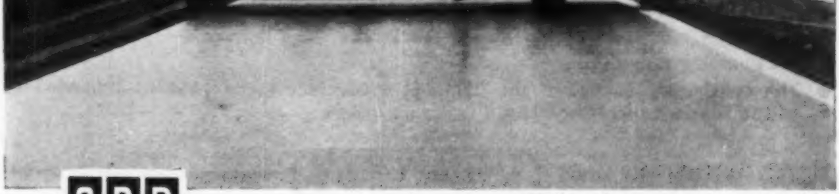

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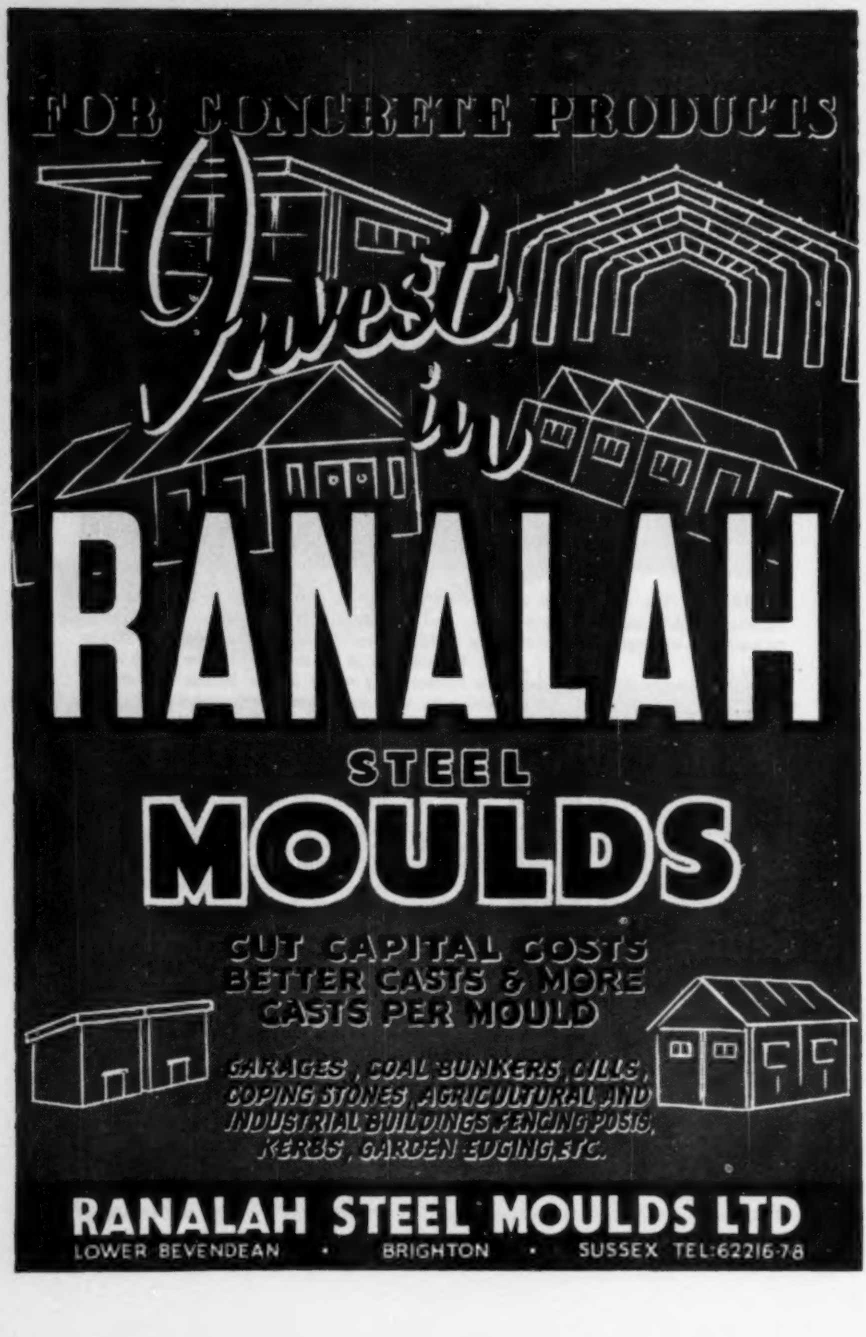
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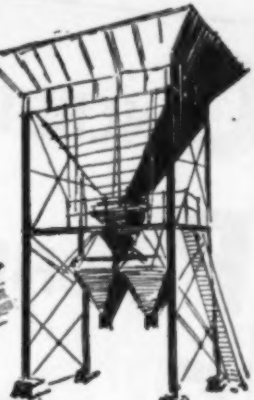

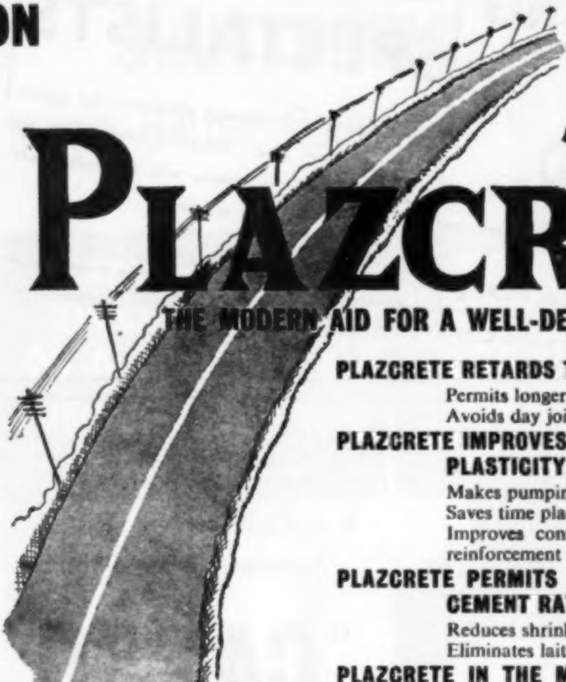

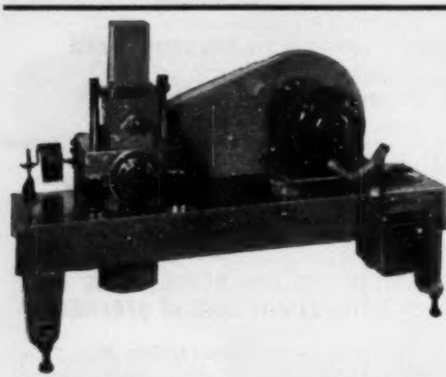

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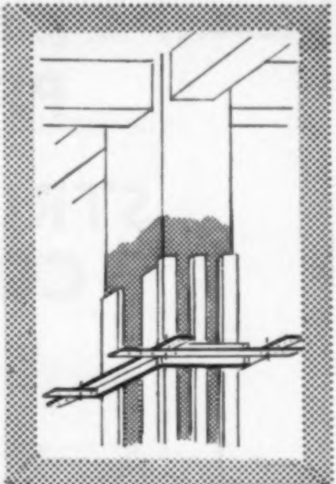

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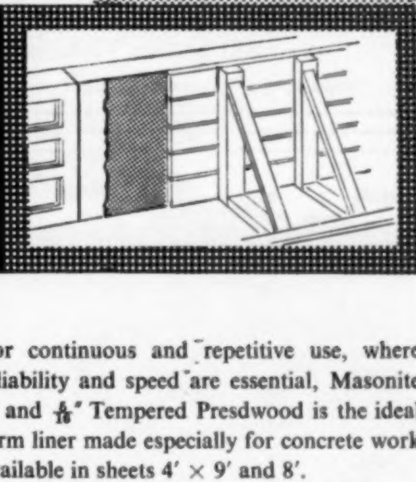

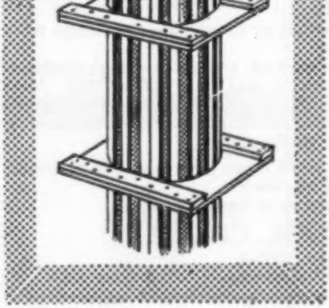

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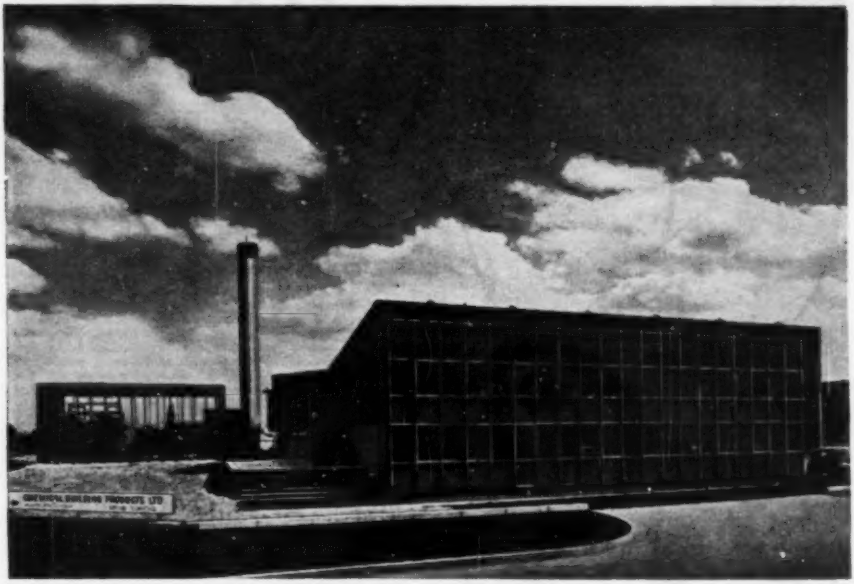

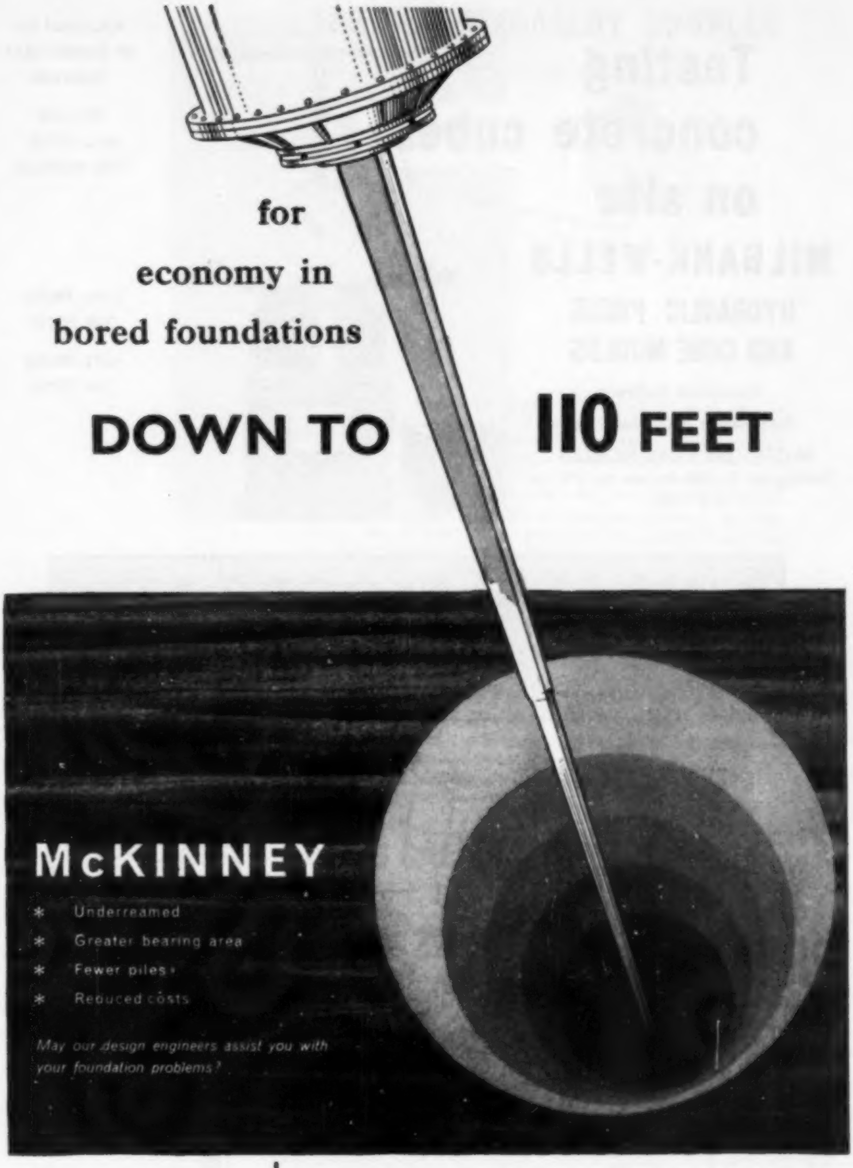

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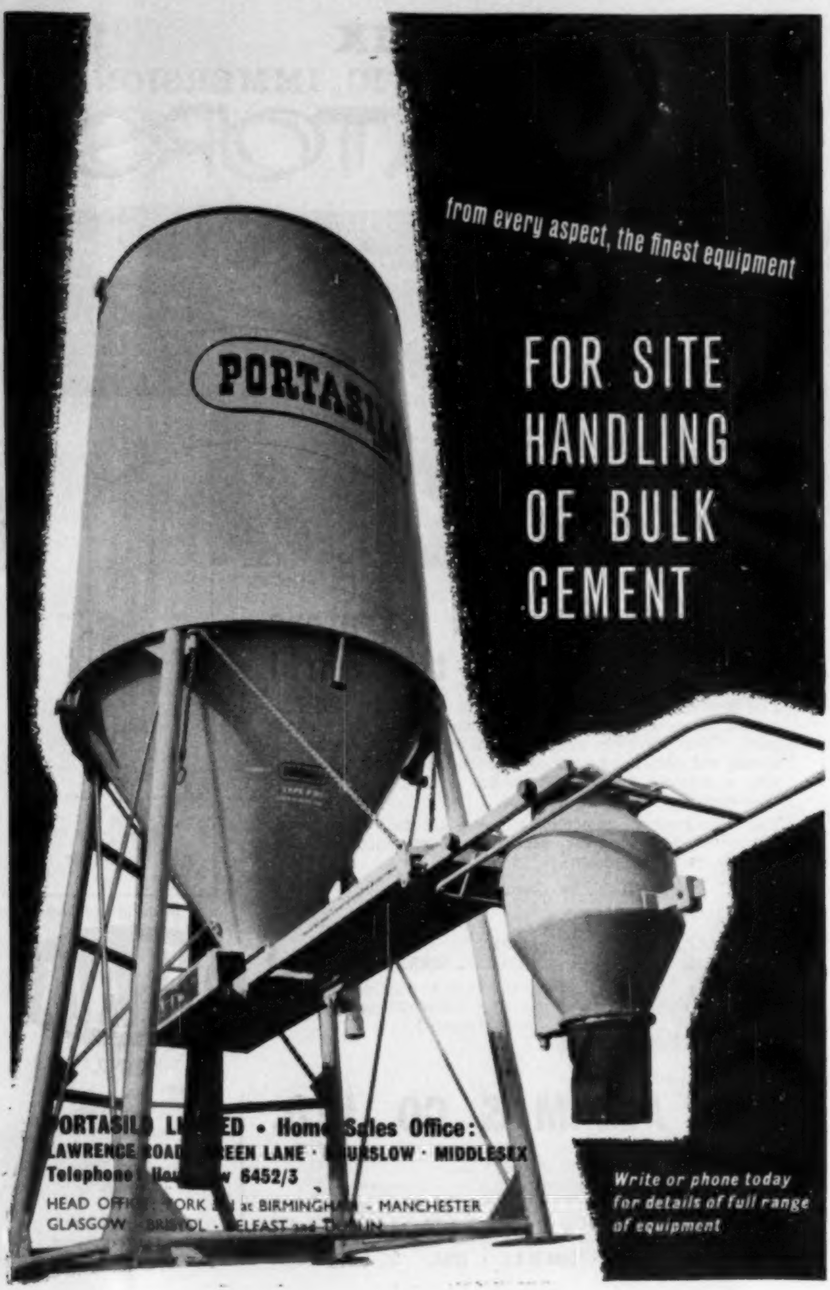
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CONCRETE AND CONSTRUCTIONAL ENGINEERING
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Volume LVI, No. 6. London, June, 1961.

EDITORIAL NOTES

Speed of Construction.

A FEATURE of current civil engineering works and buildings in Great Britain is in many cases the greater speed of construction compared with that common a few years ago and, in some instances, compared with the general rate in pre-war years. That the productivity of the constructional industries is increasing is borne out by statistics issued by the Ministry of Works which show that production, which in 1959 was 11 per cent. more than in 1954, was 18 per cent. more in 1960. This increase unaccompanied by a proportionate increase in manpower is in part due to the more extensive use of mechanical aids to construction and in part due to the greater attention given to the planning of the work before work on the site commences. The extent to which mechanisation is used on constructional sites is shown by the fact that it is estimated that up to 1914 the horse-power per man was less than five whereas on motorway M1 it was about twenty and on the construction of the new Spencer Steelworks at Llanwern, Mon., which is claimed to be the largest civil engineering contract ever placed in Great Britain with one firm, the equipment is rated at about 27 h.p. per man.

Speed of construction is not unrelated to cost. One form of construction may be more expensive than another but the extra cost may be offset by financial savings or gains resulting from the shorter time taken to erect the structure. But if greater speed of construction is allied to the cheaper form of construction, then obviously that form is the most advantageous. In a survey, made recently by the Reinforced Concrete Association, of nine reinforced concrete multiple-storey buildings in various parts of Great Britain, it is established that the average time to construct a floor was fifteen days or less, and the average rate for each of the floors above the second floor was twelve days or less. Generally office buildings were in the higher range (that is, more than a working week for each storey), while residential buildings were among the lower range, some such structures requiring one to two days only for each floor. There appears to be reliable documentary evidence that there is at present a saving of from five to ten per cent. of the total cost of a building, and at least twenty per cent. of the cost of the structural frame, if constructed in structural concrete compared with steelwork encased in concrete.

However, the average rate of construction for each floor of a building may not

be a measure of the overall time of construction of a complete building as it is well known that the foundations and other operations preliminary to the construction of the superstructure and, more particularly, the finishing operations protract considerably the period before a building can be put into use. Although the frame and floors of an eleven-storey building in London was constructed in eighteen weeks, the building took ten months in all to complete; nevertheless a good performance. Another noteworthy example of overall speedy construction was a new eighteen-storey hotel in London, which cost about £2,000,000 and gained commendation by the American owners since the work was completed in appreciably less time than they would expect it to take in the U.S.A. In fact this building, which is one of the tallest residential structures in London and the volume of which is about 2,500,000 cu. ft. and which contains about 13,500 cu. yd. of concrete, was constructed in eighteen months, that is at the overall average rate of a storey per month, although the actual rate per floor was considerably less than this. The overall time was said to be six months less than the scheduled period. An office building in London also comprising in part eighteen storeys was also erected in eighteen months. Other examples have been given in this journal.

Comparison of estimated and actual times of construction may not always be valid because the reason for the latter being the shorter may be that the estimated time was unduly pessimistic or that the amount of time lost by bad weather was less than expected. There are cases, however, in which comparative times are valid because of speeding up of operations due to good design and planning or mechanisation; or it may be because a special method of construction is adopted, as in the construction of the trial tunnels of the proposed Victoria–Walthamstow tube railway, which is described elsewhere in this number and for which the rate of boring and lining was twice that of similar tunnels constructed by the methods used before the war. Other methods, such as constructing the minimum amount of foundations to support the dead load of the structure or of the lower floors only so that construction of the upper floors and finishing operations can proceed while the basement and foundations are being completed, or construction by the lift-slab method by which delays by inclement weather are largely avoided, or construction completely or partially of precast concrete are some of the means whereby work on the site is accelerated. Or more unusual methods may be adopted as in the case of a departmental store in Glasgow in which, after the erection of the frame, the roof and then the upper floors were constructed successively from the top downwards while construction of the lower floors proceeded successively upwards.

The effect of design on the speed of construction was the subject of a lecture organised by the National Federation of Building Trade Employers and given last autumn to the Royal Institute of British Architects by a representative of a well-known firm of contractors. It was stated that the architect exerts considerable influence on the constructional time by virtue of the wide choice of materials and designs available, but any potential saving of time due to a well selected design can be offset by delays in the receipt of information on the site. Standardisation of the sizes of structural parts, whereby the greatest number of uses of the shuttering is obtainable, is desirable; the total time lost in altering shuttering may add from four to eight weeks to the constructional time of a multiple-storey building. It was stated that the speed of construction of a reinforced concrete

frame is determined by the shuttering gang rather than by the concretors. While in this country drier mixtures of concrete tend to be used, with accompanying increase in the difficulty of placing, practice in the U.S.A. seems to favour concrete in superstructures having a slump of 3½ in. to 5 in., which may expedite placing, but nevertheless the time spent on the substructure and superstructure is in general much the same in both countries. The saving in time in the U.S.A. is generally on the cladding and finishes which may be a little more austere than is usual in Great Britain.

Precasting can do much to accelerate construction if the parts are cast sufficiently in advance of the requirements on the site, and some creditable performances have been recorded. There is the case of a small laundry building, the precast frame of which was erected within eight days of the receipt of the order, the entire building taking eleven weeks. At a factory near Liverpool, which cost £2,000,000 and was built in twelve months, one building 123 ft. wide incorporated precast frames the erection of which commenced six weeks after placing the order and proceeded at an average rate of three bays (including the roof) each week, the maximum rate being five bays per week. Such performances, good as they are, do not compare with what is probably the record, namely the erection in 1959 of a five-storey block of flats in France. The cast-insitu foundations were constructed within a month; nine days later the superstructure, which comprised large precast elements, was completed; the tenants were in occupation of the building within eight weeks from the start of building operations.

Design of Reinforced Concrete Members.

Methods of designing and analysing reinforced concrete members are very numerous and it may be thought that there is no new manner of presenting aids to rapid design. In this number is given the first of a series of articles describing a method which can claim to be most comprehensive in so far that it applies to members of any cross-section, whether subjected to direct force or simple bending, or to bending combined with direct force. The basis is in accordance with the elastic (modular-ratio) or load-factor methods and is applicable whether the design has to comply with the British Standard Code, the Code of the American Concrete Institute, or similar regulations whatever, in the case of load-factor methods, the load factor may be.

The principle is that the resistance of members of any cross-section can be determined by the same procedure of computation by introducing non-dimensional factors relating to the shape, applied moment and direct force, and stresses. In the series of articles it is assumed that the section has an axis of symmetry in the plane of the load, and that the tensile strength of the concrete is ignored. Only cases where compressive and tensile stresses are developed simultaneously are considered. The method is developed for a general form of distribution of compressive stress in the concrete and is then applied in detail to the particular distributions assumed in the elastic and the load-factor theories. The method is readily applied by means of nomograms of non-dimensional form with all the quantities involved considered as variables, enabling designers to carry out accurate calculations easily and rapidly. The application of the method and the use of nomograms is illustrated by reference to a general trapezoidal section, particular cases of which are rectangular and triangular sections but nomograms for members of other cross-sections and for the case where the concrete is effective in tension and where the stresses are compressive only have been prepared.

In presenting charts, the form likely to be accepted by designers should be considered but, as has been pointed out before in this journal, nomograms have unique features when more than three parameters are involved.

Book Reviews.

"Frames and Arches". By V. Leontovich. (London: McGraw-Hill Publishing Co., Ltd. 1959. £7 15*s*.)

THIS is an important book and should be of considerable practical aid to the designer of indeterminate structures. To quote from the foreword contributed by Prof. S. P. Timoshenko: "This book is a considerable step forward in applied structural engineering. Its contents are selected to facilitate practical design of rigid frames and arches. For the first time, a solution for analysis of frames with members of variable cross-section, based on the classical method, is presented in a convenient practical form. All computations with this method are confined to simple, short operations; the time required for frame analysis is greatly reduced. Another important feature of the book is a new solution of frames with members of constant cross-section. Since the author's method is substantially shorter than any other in current practice, his approach should, no doubt, be favoured by designers."

These claims are not extravagant. Comparisons with the better-known "Kleinlogel" are unavoidable since the formulæ are presented in a very similar manner. The book under review is limited to structures of one bay or span, and to frames the two legs of which are of the same length, and in these respects is not so fully comprehensive. It does, however, include frames each member of which is of non-uniform cross-section and, more importantly, frames with haunches or splays in the corners. The shapes of the hinged and fixed frames dealt with are rectangular portal frames with straight and parabolic beams, trapezoidal frames and gable frames. The consideration of arches is limited to fixed and two-hinged parabolic forms.

"A Guide to the B.S. Code of Practice for Prestressed Concrete." By F. Walley and S. C. C. Bate. (London: Concrete Publications, Ltd. 1961. Price 12*s*. 6*d*.; by post 13*s*. In Canada and U.S.A. 3 dollars.)

THIS book provides an informed commentary on the British Standard Code of Practice CP. 115 (1959), "The Structural Use of Prestressed Concrete in Buildings", since the authors were prominent members of the committee responsible for the compilation of the Code. The Code is given in full and the comments and explanations follow each clause. Recommendations relating to aspects of prestressed concrete construction which are not common to other forms of concrete construction are in particular amplified. The subject matter is dealt with under the same headings as in the Code, namely definitions and symbols, materials, design stresses, the design of structural members subjected to pure bending or direct compression, fire resistance, methods of prestressing, practical considerations, inspection and testing. A concise bibliography is given and there are twelve tables in addition to those in the Code, and twelve diagrams or charts containing valuable data for the designer particularly in respect of eccentrically-loaded members. The commentaries on steel for prestressing, stresses, loss of prestress, resistance to shearing, the ultimate resistance of beams, and fire resistance are noteworthy in respect to their length and content.

"Building Construction Estimating." By G. H. Cooper. (London: McGraw-Hill Publishing Co. Ltd. 1959. Price 60*s*.)

THIS second edition of an American book on the subject of cost estimating for buildings is an enlargement of the preceding edition. It is essentially a textbook for students learning the principles of the subject rather than a guide giving prices and data for a contractor's estimator. It is therefore of value to students other than those concerned solely with American practice. The scope of treatment is limited mainly to house building and therefore reinforced concrete and allied methods of construction are not dealt with fully except in so far that foundations, floors, and walls are built of concrete. The range of topics is quite wide, extending as it does from the relations between architect, contractor and client to the commercial sizes of nails. Some interesting sidelights are also included such as the diagram illustrating how a tree is sawn to provide timber for various constructional purposes.

A Practical Comprehensive Method of Designing Reinforced Concrete Sections.—I.

By J. RYGOL, B.Sc.(Eng.).

WHEN designing or analysing reinforced concrete sections of various shapes the use of a method of computation, which is applicable to axial or eccentric compression or tension, and simple bending, and is in accordance with the elastic and the load-factor methods, is advantageous. Such a method is described in these articles. (See Editorial Note on page 207.)

The problem involves the consideration of three interrelated factors (*Fig.* 1), namely (a) the details of the section, (b) the loading, and (c) the stresses developed in the section when subjected to the loading. The principal dimensional quantities which describe these factors are interrelated in one set of basic equations using non-dimensional factors referred to as stress-factors. The general equations of equilibrium of the internal stresses and the external loading are then expressed in non-dimensional form in terms of the stress-factors, and these equations are referred to as the stress-factor equations.

DIMENSIONAL QUANTITIES.

Section.

d = effective depth; h = overall depth; t = depth dimension.
b = reference breadth; b_0 = breadth dimension.
x = depth to neutral axis.
a = depth of equivalent rectangular stress block at ultimate load.
g, g' = concrete cover of tension and compression reinforcement respectively.
A_s, A_s' = cross-sectional area of tension and compression reinforcement respectively.
I = moment of inertia of transformed concrete section.
A_c = cross-sectional area of concrete in compression.

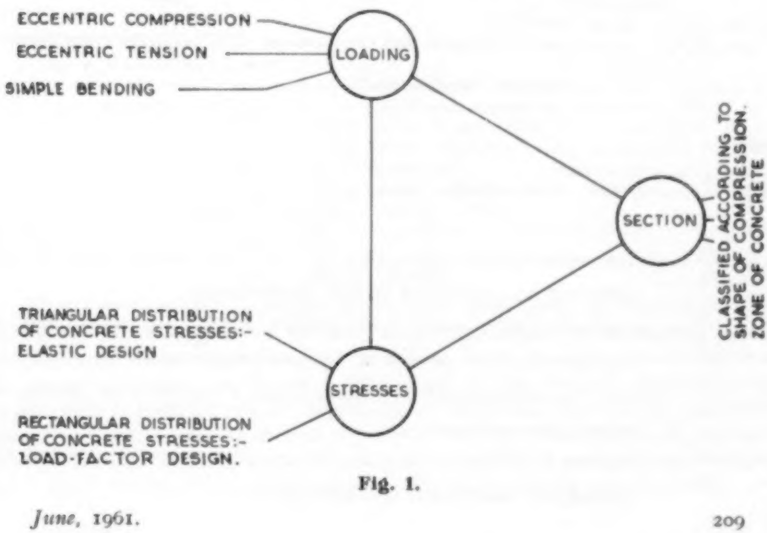


Fig. 1.

Loading.

N = applied normal force (compressive or tensile).

$N_u = kN$ = ultimate normal force.

e_t, e_c = eccentricity of N (or N_u) measured from centroid of tension and compression reinforcement respectively.

$M_t = Ne_t$; $M_c = Ne_c$.

$M = M_t = M_c$ = applied bending moment when $N = 0$.

ΔM_t = unbalanced part of bending moment M_t in $M_t = \bar{M}_t + \Delta M_t$.

$M_{ut}(= N_u e_t)$, $M_{uc}(= N_u e_c)$, $M_u(= M_{ut} = M_{uc})$, and $\Delta M_{ut}(= M_{ut} - \bar{M}_{ut})$ = values of M_t, M_c, M, and ΔM_t respectively at ultimate load.

Stresses and Strains.

f_c = maximum compressive stress in concrete (elastic).

f_s, f_s' = stress in steel at centroid of tension and compression reinforcement respectively (elastic).

f_u = uniform stress in concrete of equivalent rectangular stress-block at ultimate load.

f_y, f_y' = yield-point stress of tension and compression reinforcement respectively.

u, u_{cyl}, u_{pr} = 28 days' strength of 6-in. concrete cubes, 6-in. by 12-in. cylinders, and prisms, respectively.

D_c = resultant of compressive stresses in concrete.

D_{cu} = resultant of compressive stresses in concrete at ultimate load.

E_s, E_c = modulus of elasticity of steel and concrete respectively.

ε_c = maximum compression strain of concrete (elastic).

ε_s, ε_s' = strain (elastic) in tension and compression reinforcement respectively.

ε_u = maximum compressive strain of concrete at ultimate load.

ε_y, ε_y' = yield strain of tension and compression reinforcement respectively.

NON-DIMENSIONAL RATIOS.

Section.

$\delta = t/d$ = depth-ratio; $\gamma = b_0/b$ = breadth-ratio; $n = x/d$ = neutral-axis depth-ratio.

$\alpha = a/d$ = depth-ratio of equivalent rectangular stress block at ultimate load.

$\beta = g/d$, $\beta' = g'/d$ = embedment-ratio of tension and compression reinforcement respectively.

$\nu = A_c/(bd)$ = shape coefficient.

$p = A_s/(bd)$, $p' = A_s'/(bd)$ = tension and compression reinforcement-ratio respectively.

$\lambda = z/d$, $\lambda' = z'/d$ = lever-arm depth-ratios.

$\mu = I/(bd^3)$ = moment of inertia coefficient.

Loading.

$k = N_u/N$ = load factor.

$\varepsilon = e_t/d$; $\varepsilon' = e_c/d$ = eccentricity ratios.

Stresses.

$q = p\dfrac{f_s}{f_c}$ = tension reinforcement-index (elastic), $= p\dfrac{f_s}{f_u}$ at ultimate load, if not controlled by yielding of tension reinforcement.

$q' = p'\dfrac{f_s'}{f_c}$ = compression reinforcement-index (elastic), $= p'\dfrac{f_s'}{f_u}$ at ultimate load, if not controlled by yielding of compression reinforcement.

$q_y = p\dfrac{f_y}{f_u}$ = tension reinforcement-index at ultimate load if controlled by yielding of tension reinforcement.

$q_y' = p'\dfrac{f_y'}{f_u}$ = compression reinforcement-index at ultimate load if controlled by yielding of compression reinforcement.

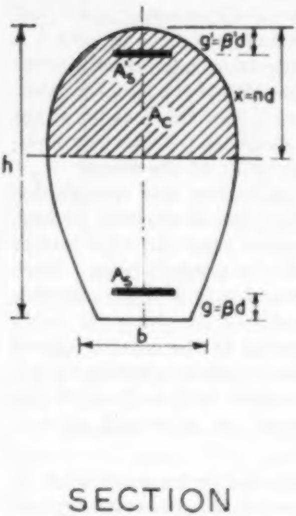


Fig. 2.

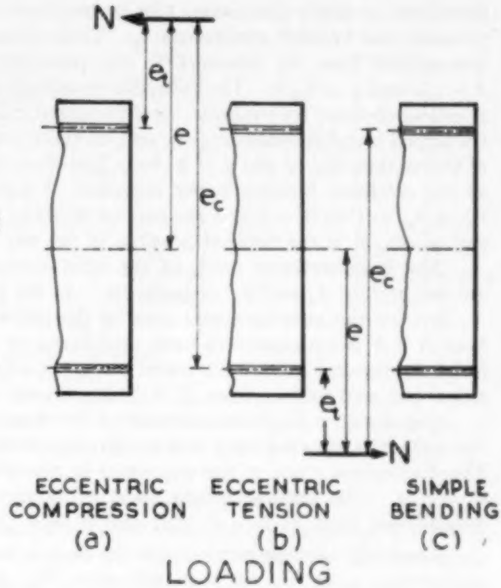


Fig. 3.

$r = \dfrac{f_s}{f_c}$ = stress-ratio (elastic); $r_y = \dfrac{f_y}{f_u}$ = stress-ratio at ultimate load.

$m = E_s/E_c$ = modular ratio; $m_u = (E_s e_u)/f_u$ = plastic modular ratio at ultimate load.

Non-dimensional Factors.

$\kappa_1, \kappa_2, \kappa_3$ = stress-block factors.
$\kappa_{1u}, \kappa_{2u}, \kappa_{3u}$ = stress-block factors at ultimate load.
ϕ_1, ϕ_2, ϕ_3 = shape factors.
K_c, K_s = concrete and steel stress-factor respectively for normal force N.
C_t, S_t = concrete and steel stress-factor respectively for moment of N about centroid of tension reinforcement.
C_c, S_c = concrete and steel stress-factor respectively for moment of N about centroid of compression reinforcement.
K_u, C_{tu}, C_{cu} = values of K_c, C_t, and C_c respectively at ultimate load.

Basic Ratios.

Shape of Section.—A reinforced concrete section (*Fig.* 2) is described completely by the dimensions of the secuon and the amount and location of the reinforcement. The classification of sections is made according to the shape of the compression zone of concrete. The shape of the concrete section in the tension zone is generally of no importance as the tensile strength of concrete is ignored. The principal dimensions of a section are the effective depth d, measured from the extreme edge in compression to the centroid of tension reinforcement, and a freely chosen reference breadth b. The shape of the compression zone is

described by depth dimensions t (measured from the same extreme edge in compression) and breadth dimensions b_0. These dimensions can be expressed in non-dimensional form by reference to the principal dimensions d and b as ratios $\delta = t/d$ and $\gamma = b_0/b$. The reference breadth b and the dimensions t and b_0 are practical choices; for example, for a rectangular section the reference breadth b is the actual breadth of the section, and, as there is no variation in depth or breadth of the section, $\delta = 0$ and $\gamma = 0$; for a T-section the breadth of the flange is taken as the reference breadth b, the thickness of flange as t, and the breadth of the rib as b_0, so that $\delta(= t/d)$ is the ratio of depth of the flange to the effective depth and $\gamma(=b_0/b)$ is the ratio of breadth of the rib to breadth of the flange.

The cross-sectional areas of the reinforcement in tension and compression are denoted by A_s and A_s' respectively. In the general case of eccentric loading, A_s denotes the cross-sectional area of the reinforcement near the edge farther from N if N is a compressive force, and nearer to N if N is a tensile force. Similarly, A_s' denotes the reinforcement along the edge nearer to N if N is a compressive force, and farther from N if N is a tensile force.

The distance from the centroid of the reinforcement to the nearest edge of the section is denoted by g when referring to A_s, and g' when referring to A_s'. The dimensions g and g' are expressed in non-dimensional form as $\beta = g/d$ and $\beta' = g'/d$. The cross-sectional areas of reinforcement are expressed in non-dimensional form as $p = A_s/(bd)$ and $p' = A'_s/(bd)$.

Loading.—In the general case the section is subjected to a normal force N, compressive as in *Fig.* 3a, or tensile as in *Fig.* 3b, acting at an eccentricity e if measured from the mid-axis of the section, and e_t and e_c if measured from the centroids of the tension and compression reinforcements respectively. The moments of N about the tension and compression reinforcements are $M_t = Ne_t$ and $M_c = Ne_c$. The eccentricities e, e_t, and e_c are expressed as ratios of the effective depth by $\varepsilon_0 = e/d$, $\varepsilon = e_t/d$, and $\varepsilon' = e_c/d$. Simple bending

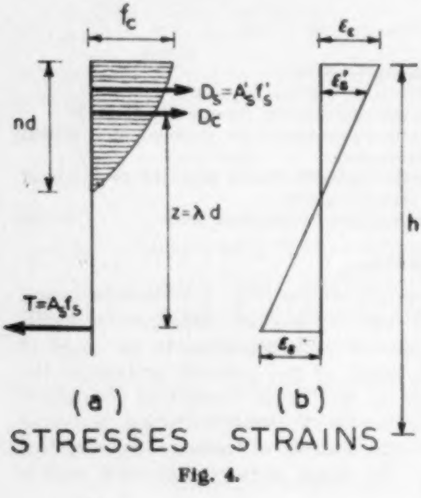


Fig. 4.

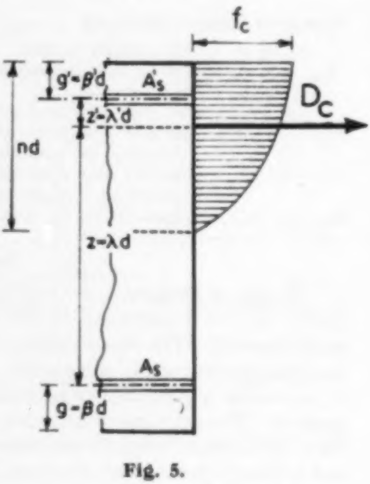


Fig. 5.

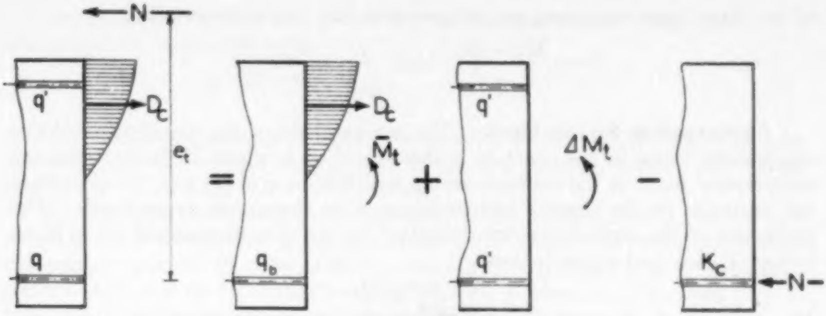


Fig. 6.
(*Note.*—$q_b = \bar{q}_b$.)

($M = M_t = M_c$) as in *Fig.* 3c is a particular case of eccentric loading when $e = e_t = e_c = \infty$. At ultimate load, the normal force is N_u, and the moments of N_u about the centroids of tension and compression reinforcement are M_{ut} and M_{uc} respectively.

Stresses.—In the general case the distribution of stress in the compression zone is curvilinear (*Fig.* 4a). The principal stresses are the compressive stress f_c at the extreme edge, and the stress f_s in the tension reinforcement. The stress in the compression reinforcement is f_s'. The ratio of f_s to f_c is denoted by r, and the ratio of f_s' to f_c by r'. At ultimate load the compressive stress in the concrete is f_u and the stress in the tension reinforcement attains usually the yield-point stress f_y (f_y' in the compression reinforcement); hence $r_y = f_y/f_u$ and $r_y' = f_y'/f_u$.

Strains.—Plane sections before loading are assumed to remain plane after loading (*Fig.* 4b), that is, the strain at any element of the section is proportional to the distance of the element from the neutral axis. The strain at the extreme edge in compression is ε_c, in the tension reinforcement ε_s, and in the compression reinforcement ε_s'. At ultimate load the maximum strain of the concrete is ε_u, in the tension reinforcement ε_y, and in the compression reinforcement ε_y' if controlled by yielding of the steel.

Basic Equations and Factors.

The basic equations, expressed in terms of the concrete stress f_c, are

$$N = K_c f_c bd, \qquad (1)$$
$$M_t = Ne_t = C_t f_c bd^2, \qquad (2)$$
$$M_c = Ne_c = C_c f_c bd^2, \qquad (3)$$

where K_c, C_t, and C_c are the stress-factors. The sign convention in respect of N is as follows. In equation (1) N is positive if it is compressive and is negative if tensile; hence K_c is positive if N is compressive and negative if N is tensile. In equations (2) and (3) N is entered at its numerical value; the eccentricity e_c is negative if the line of action of N is between A_s' and A_s, and positive if outside.

As the equilibrium between the internal stresses in the section and the external loading is described completely by two equations, it follows that only two

of the three basic equations are independent, the interrelation being

$$N = \frac{M_t - M_e}{d - g'} \quad \text{and} \quad K_e = \frac{C_t - C_e}{1 - \beta'}.$$

Compressive Stress-block.—The general form of the distribution of the compressive stress in the concrete is shown in *Fig.* 5, where D_c is the resultant compressive force in the concrete acting at distances $z(= \lambda d)$ and $z'(= \lambda' d)$ from the centroids of the tension and compression reinforcement respectively. The properties of the stress-block are described by the non-dimensional stress-block factors κ_1, κ_2, and κ_3 as follows.

$$D_c = \kappa_1 f_c bd. \qquad (4)$$

The moment of D_c about the centroid of tension reinforcement is

$$zD_c = \kappa_2 f_c bd^2. \qquad (5)$$

The moment of D_c about the centroid of compression reinforcement is

$$z'D_c = \kappa_3 f_c bd^2. \qquad (6)$$

The relation between the factors κ_1, κ_2, and κ_3 is

$$\kappa_1 = \frac{\kappa_2 + \kappa_3}{1 - \beta'}. \qquad (7)$$

Thus the lever-arm depth-ratios of D_c about the centroid of tension and compression reinforcement respectively are

$$\lambda = \frac{\kappa_2}{\kappa_1} \quad \text{and} \quad \lambda' = \frac{\kappa_3}{\kappa_1}.$$

Stress-factor Equations.—The stress-factor equations describe in non-dimensional form the equilibrium conditions between the internal stresses and the external loading, and are

$$K_e = \kappa_1 + q' - q, \qquad (8)$$
$$C_t = \kappa_2 + (1 - \beta')q', \qquad (9)$$
$$C_e = -\kappa_3 + (1 - \beta')q, \qquad (10)$$

where $q = pr$ and $q' = p'r'$, which are the tension reinforcement-index and the compression reinforcement-index respectively.

Simple Bending.—For simple bending $N = 0$ and $M_t = M_e = M$; hence $K_e = 0$ and $C_t = C_e = C$, and the equation (8) becomes

$$\kappa_1 + q_b' - q_b = 0$$

in which the subscript b indicates that the relationship is applicable to simple bending only.

Section Reinforced for Tension Only.—Since $q' = 0$, $\bar{K}_e = \kappa_1 - \bar{q}$, and $\bar{C}_t = \kappa_2$. Therefore

$$N = \bar{K}_e f_c bd, \qquad (11)$$
$$\bar{M}_t = Ne_t = \bar{C}_t f_c bd^2, \qquad (12)$$

where $\bar{K}_e$, $\bar{C}_t$, $\bar{q}$, and $\bar{M}_t$ refer to a section reinforced for tension only.

Section Reinforced for Tension and Compression.—The values of q and q' can be computed from

$$q' = \frac{C_t - \bar{C}_t}{1 - \beta'} \quad \text{and} \quad q = \bar{q}_b + q' - K_e,$$

where $\bar{q}_b(= \kappa_1)$ is the tension reinforcement-index of the section, with the same stress-block, reinforced for tension only to resist a bending moment $\bar{M}_t$. Note that $M_t = \bar{M}_t + \Delta M_t$ and $C_t = \bar{C}_t + \Delta C_t$.

The expediency of using the foregoing relations is illustrated in *Fig.* 6 which shows how the calculation of reinforcement for a section reinforced for tension and compression can be reduced to the much simpler calculation of (i) $\bar{q}_b$ as for a section subjected to a bending moment $\bar{M}_t$; (ii) q' resisting ΔM_t; and (iii) K_e resisting N applied at the centroid of the tension reinforcement.

Sizewell Nuclear Power Station.

Construction began in April last of the nuclear power-station at Sizewell, Suffolk, for the Central Electricity Generating Board. This station will be the largest under construction in the world since, on completion in 1966, the capacity will be 580 megawatts. It is claimed that it will also be the most compact station, and it will be the first to have two reactors installed in a composite building. Some notes on, and an illustration of, the station are given in this journal for January 1961.

The 400-ton Goliath crane at the nuclear power-station at Hinkley Point is to be removed and re-erected at Sizewell early next year. There will also be two 7-ton ground-derricks with 120-ft. jibs. Gabbard-derricks and tower-cranes will be used in the construction of the biological shields and the superstructure of the reactor building. Construction of the foundations involves more than 500,000 cu. yd. of excavation. It is expected that the concrete batching and mixing plants will produce concrete at a rate of up to 4000 cu. yd. weekly.

To minimize differential settlement, comparable operations on the two reactors will take place within a short interval of time. The foundations of the reactors will be a reinforced concrete raft 225 ft. long and 110 ft. wide, and will be constructed in a series of lifts and bays, and will require the excavation of 17,000 cu. yd. of sand and the placing of 140,000 cu. yd. of concrete. The superstructure of the reactor building will be 193 ft. high and 400 ft. long. The primary biological shield-walls around the pressure vessels will be duodecagonal on plan, and will be of concrete 6 ft. thick and will be more than 86 ft. high above the foundations.

The foundations of the turbine house will be deeper than those of the reactors, and will require excavation to a maximum depth of 35 ft. About 140,000 cu. yd. of sand will be excavated and about 130,000 cu. yd. of concrete is to be placed below ground level. An extensive dewatering system will be required to lower and control the ground water.

The cooling-water intake will be 1800 ft. off-shore and sea-water will be drawn through this installation at the rate of over 27,000,000 gallons an hour. After use the water will be discharged at a point 500 ft. off-shore. Four tunnels of 10 ft. diameter will be driven in compressed air in connection with the off-shore works. A major dewatering scheme will be required for constructing the cooling-water pumphouse and access shaft to the tunnels. Excavations will extend to a depth of about 70 ft. and the water-table will be lowered by about 40 ft. The access shaft will be formed within a coffer-dam of steel sheet-piles. The excavation for the pumphouse will be carried out within a system of open-cut excavations and steel sheet-piles. Tests of alternative procedures of sheet-piling have been carried out to determine the optimum method for driving the piles.

The labour force is expected to be about 2400 in 1962. The station is being constructed by the Atomic Power Group comprising The English Electric Co., Ltd., Messrs. Babcock & Wilcox Ltd., and Taylor Woodrow Construction Co., Ltd.

Fig. 1.

(*See facing page.*)

A Concrete-lined Tube-railway Tunnel.

SOME experimental tunnels for tube railways are being constructed by London Transport in north London. One of the objects of the work is to enable experience of new methods of tunnel construction to be obtained prior to the commencement of the construction of the new tube railway from Victoria to Walthamstow; the experimental tunnels are on the alignment of the new railway and will be incorporated therein. The new railway will be 11½ miles long. The experimental tunnels comprise a mile length of double tunnel extending from Finsbury Park to Netherton Road, Tottenham. Another object is to determine whether cast iron segments or precast concrete are more suitable for lining tunnels bored in London clay by a new type of machine. The linings are not bolted or grouted. About ½ mile of double tunnel with cast-iron segments and a ½ mile of single tunnel with concrete blocks (*Fig.* 1) have been completed. A feature of the work is that tunnelling proceeds at about twice the rate attained by methods previously used.

Excavation.

The tunnels are being bored by a type of rotary shield called a "drum digger", which can work much faster than any previous type of shield used for tube-railway construction in London and which makes provision for belt-conveyors to remove the spoil.

There are working sites and shafts at the corner of Netherton Road and Seven Sisters Road (Tottenham), and in Finsbury Park. The shaft at Netherton Road is 25 ft. in diameter and 60 ft. deep, and will be a permanent part of the new railway since it will be used for ventilation and emergency stairs. From the bottom of the shaft an access tunnel extends a short distance to the line of the rail tunnels, where chambers large enough to permit the assembly of the rotary shields have been excavated by pneumatic spades and lined with bolted cast-iron segments. At this site, excavated spoil is brought to the surface by a crane which lifts 1¼-cu. yd. skips off the 2-ft. gauge bogies which have brought the spoil, in trains hauled by battery locomotives, along the completed section of the tunnels. The skips are loaded by belt-conveyors fed from the rotary cutters at the working face, and are emptied on to the ground at the surface, where the spoil is loaded into lorries by a 1-cu. yd. mobile shovel.

At the Finsbury Park site, the shaft is 15 ft. in diameter and 60 ft. deep. As at the other site, an access tunnel leads from the bottom of the shaft to the rail tunnels, and there are two chambers in which the shields are assembled. The rail tunnels are on a gradient of 1 in 47 and the loaded ⅞-cu. yd. skips are hauled by an electric winch located in the shield-assembly chamber. The spoil is tipped from the skips on to a belt-conveyor in the access tunnel and the conveyor delivers the spoil to the bottom of the shaft, where it is raised to the surface by a vertical bucket-elevator. The buckets are warmed by hot air to prevent the clay from sticking to the metal. Originally, the clay was deposited in an elevated hopper under which lorries could be loaded by gravity, but because of the rate of excavation it is necessary to deposit the spoil on the ground and use a 1-cu. yd. mobile shovel for loading.

The "drum-digger" type of shield is the same as that used previously on the tunnel from Hampton to Chingford for the Metropolitan Water Board (see this journal for April, 1960), but the experimental railway tunnels are larger than the water tunnel, and the shield is therefore also larger. The "drum-digger" for the concrete-lined tunnels has an external diameter of 14 ft. and that for the cast-iron-lined tunnels 13 ft. 1 in. The machine consists essentially of two drums; that for the 14-ft. diameter tunnels has an outer drum of 14 ft. external diameter and 9 ft. long and the leading end is bevelled to form a cutting edge. Within the main drum there is a 7-ft. 6-in. diameter rotating drum, 5 ft. 6 in. long, carried on two roller races and provided with a thrust ring to take the axial load from the rotating cutters. The cutting teeth are mounted on six arms at the front of the shield, each arm having up to eight removable teeth, mounted at the outer edge of the rotating drum, so that they cut the ground in an annular area in front of the space between inner and outer drums,

The ground in front of the inner drum is cut by teeth mounted on a removable arm across the inner drum. The inner drum and cutting teeth are rotated at speeds of up to 4 revs. per minute by six hydraulic motors.

The forward movement of the shield is effected by fourteen hydraulic rams arranged at equal intervals around the periphery of the shield and pushing against the last completed ring of lining. The rams, which have a stroke of 2 ft. 8 in., are controlled separately by an operator standing behind the rotating inner drum within the shield casing. The operator is provided with sighting guides which show whether the shield is on the correct line and level and he can increase or reduce the pressure on any of the rams to correct any tendency of the shield to deviate from the correct alignment. The clay is cut by the teeth, and by means of scoops and paddles it is guided into a hopper within the shield where it drops on to the end of an inclined belt-conveyor, which discharges on to the main horizontal belt-conveyor on which the excavated clay is carried along a staging to be discharged into skips running on a 2-ft. gauge track on each side of the conveyor. The whole of this equipment including the staging is mounted on an articulated trailing platform which is attached to the rear of the shield and is drawn forward as the shield advances. When the rams have pushed the shield forward a distance equal to the width of a ring of lining, that is 2 ft., they are retracted. A ring of lining is then placed in the space between the last completed ring and the rear of the shield. When this has been completed the cycle is repeated, the rams thrusting against the newly-placed ring.

The "drum digger" is capable, in good conditions, of advancing more than 60 ft. in three 8-hour shifts, over long periods. The greatest length of tunnel driven to date in 24 hours is 88 ft.

Concrete Lining.

The precast concrete blocks for the lining are of a new type. The tunnel is bored to provide for a tube of 12 ft. 6 in. internal diameter with a lining of concrete blocks. Blocks of various thicknesses from 4½ in. to 9 in. have been tried. The thickness likely to be used eventually will be 6 in. Each tunnel ring is made up of fourteen identical blocks each having one cross-joint face convex and the other concave, so that they fit together with knuckle joints. When the fourteen blocks are assembled a gap of about 7 in. remains at the top, and is filled by a pair of reinforced concrete folding wedges having plane contact faces with concave and convex faces respectively in contact with the blocks on each side. The wedge, with the wider end towards the shield, is driven home by a pair of small hydraulic rams while the other wedge is held in position. The two wedges hold the whole ring firmly in place. The wedges are seen in place at the top of *Fig.* 1.

The blocks have four holes spaced equally around the inner face and into which bolts are fixed for handling the blocks in the tunnel and which will also be used for fixing permanent tunnel equipment. The blocks are handled by hoists at the working face, an expanding bolt placed in one of the holes being used to lift them. They are hoisted into their correct positions by an arm mounted at the rear of the shield, and are held in place by pull-out "needles" until the concrete wedges are driven into position. When firmly in position, the lining bears against the face of the excavation in the clay.

Precast Blocks.

The concrete blocks, which are made at a works at Waltham Abbey, are not reinforced and are of concrete having a crushing strength of not less than 5500 lb. per square inch. The mixing water is at a temperature of 140 deg. F. at the time of gauging. Ordinary Portland cement is used.

The blocks are lifted by crane, the tackle of which is attached to eye-bolts let into holes in the blocks. The convex and concave ends are coated with bituminous paint. At the site, the blocks are secured in a rope-sling, four at a time, and lowered into the shaft and placed on a train of bogies which carries in one journey the fourteen blocks required for one ring.

For the concrete-lined tunnel the consulting engineers are Sir William Halcrow & Partners. The contractors are Messrs. Kinnear Moodie & Co., Ltd. The precast concrete blocks and wedges were made by Messrs. Charles Brand & Son, Ltd.

Slabs for Liquid-containing Structures.

DESIGN IN ACCORDANCE WITH B.S. CODE OF PRACTICE No. 2007 (1960).

THE accompanying tables give data to enable the walls and other slabs in a structure containing water or other aqueous liquid to be designed quickly in accordance with the recommendations of British Standard Code No. 2007 (1960). There are three principal cases to consider, namely, (I) Slabs subjected to bending only; (II) Slabs subjected to direct tension only; and (III) Slabs subjected to bending and direct tension. *Table* A deals with cases I and II and *Table* B with case III. The following conditions are applicable to all cases.

The tensile stress p_{st} in the reinforcement shall not exceed 12,000 lb. per square inch (except in tensile reinforcement near the face remote from the liquid for slabs not less than 9 in. thick).

The nominal volumetric proportions of the concrete shall be 1 : 1·6 : 3·2 and the compressive stress p_{cb} in this concrete shall not exceed 1200 lb. per square inch; this requirement is seldom critical. The tensile stress p_{ct} in concrete of this mixture shall not exceed 190 lb. per square inch in slabs in direct tension and 270 lb. per square inch in slabs in bending; this requirement applies when it is necessary to design for resistance to cracking.

The cover of concrete over the reinforcement shall be not less than $1\frac{1}{2}$ in. or the diameter of the bar if exceeding $1\frac{1}{2}$ in.

The modular ratio m shall be assumed to be 15.

The proportion $r_0\left(=\frac{A_{st}}{bd}\right)$ of reinforcement shall be not less than 0·003 with plain bars and 0·0025 with deformed bars.

Symbols used additional to those in the foregoing are as follows,

d, d_1, b, l_a: Thickness, effective depth, breadth and lever-arm respectively of slab.

M'_{rc}, M'_{rs}: Moment of resistance when not cracked and cracked respectively based on the thickness.

Q: Moment-of-resistance factor without compression reinforcement based on the effective depth.

R_c, R_s: Factors for moment of resistance corresponding to M'_{rc} and M'_{rs} respectively.

Case I.—Slabs Subjected to Bending Only.

Slabs subjected to bending have to be designed to resist cracking and, should cracking occur, they must be reinforced sufficiently and be sufficiently thick to avoid overstressing the reinforcement in tension or the concrete in compression. The basic formulæ, which are derived from the modular-ratio method of determining the resistance, are as follows.

RESISTANCE TO CRACKING.

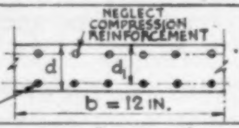
$$M'_{rc} = R_c d^2 = \frac{p_{ct}}{1 - n_0}\left[\frac{1}{3} - (1 - n_0)n_0 + r_0(m - 1)\left(\frac{d_1}{d} - n_0\right)^2\right]bd^2,$$

in which

$$n_0 = \frac{\frac{1}{2} + r_0\left(\frac{d_1}{d}\right)(m - 1)}{1 + r_0(m - 1)}.$$

TABLE A.—BENDING ONLY.

IN ACCORDANCE WITH B.S. CODE No. 2007.
CONCRETE: NOMINAL PROPORTIONS 1:1·6:3·2 $m = 15$
MINIMUM PROPORTION OF REINFORCEMENT $r_o = \frac{A_{st}}{bd} \nless 0{\cdot}003$
MINIMUM COVER: $1\frac{1}{2}$ IN.
BENDING MOMENT = M IN.-LB. PER FOOT.



CASE IA
BENDING ONLY
(TENSILE STRESS AT FACE IN CONTACT WITH LIQUID)

RESISTANCE TO CRACKING. $M'_{rc} = R_c d^2$

DETERMINES THICKNESS d. $p_{ct} \ngtr 270$ LB./SQ.IN.

GIVEN OR ASSUME r_o AND $\frac{d_1}{d}$

$$d \nless \sqrt{\frac{M}{R_c}} \quad \left[\begin{array}{l} \text{WHERE } R_c = \dfrac{3240}{1-n_o}\left[\dfrac{1}{3} - (1-n_o)n_o + 14r_o\left(\dfrac{d_1}{d} - n_o\right)^2\right] \\ \text{AND } n_o = \dfrac{\frac{1}{2} + 14r_o\left(\frac{d_1}{d}\right)}{1 + 14r_o} \end{array}\right.$$

DESIGN FOR STRENGTH. $M'_{RS} = R_s d^2$

DETERMINES REINFORCEMENT A_{st}. $p_{st} \ngtr 12{,}000$ LB./SQ.IN.

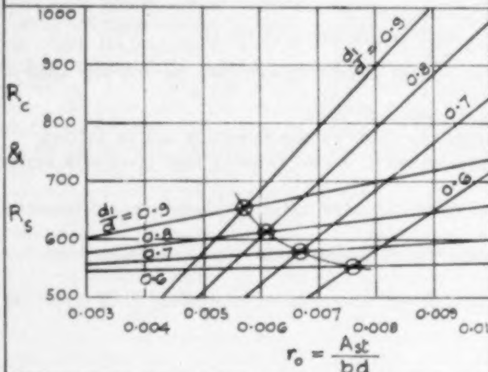
$$A_{st} \nless \frac{M}{12{,}000\, a_1 d_1} \quad \text{WHERE } a_1 = 1 - \tfrac{1}{3}n_1$$

$$\text{AND } n_1 = \sqrt{r_s^2 + 2r_s} - r_s; \quad r_s = 15r_o\left(\frac{d}{d_1}\right)$$

$$[R_s = 144{,}000\, r_o\, a_1 d_1 d]$$

BALANCED DESIGN

POINTS THUS ⊙ INDICATE VALUES OF r_{oE} AND $R_c = R_s = R_E$ FOR BALANCED DESIGN AND ARE APPLIED IN THE TABLE BELOW TO SLABS OF THE THICKNESS STATED.



d IN.	d_1 IN.	$M'_{rc} = M'_{rs}$ IN.-LB./FT.	A_{st} ϕ	A_{st} SQ.IN./FT.
4	2·25	8,800	½"	0·39
4½	2·69	10,300	⅝"	0·41
5	3·19	14,200		0·43
6	4·19	20,800		0·49
7	5·19	29,200		0·54
8	6·19	38,800		0·60
9	7·12	29,400	¾"	0·66
10	8·12	61,800		0·72
12	10·12	91,100		0·85
15	13·06	144,200	⅞"	1·05
18	16·0	211,200	1"	1·25
24	22·0	380,100		1·64

(a) $d < 9$ IN. ADOPT CASE IA.

CASE IB
BENDING ONLY
(TENSILE STRESS AT FACE REMOTE FROM LIQUID.)

(b) $d \nless 9$ IN. DESIGN FOR STRENGTH ONLY

$p_{st} \ngtr 18{,}000$ LB./SQ.IN. (PLAIN BARS) OR 20,000 LB./SQ.IN. (DEFORMED BARS)

$p_{cb} \ngtr 1200$ LB./SQ.IN.

DESIGN.

$$d_1 \nless \sqrt{\frac{M}{3000}} \text{ (PLAIN BARS) OR } \sqrt{\frac{M}{2872}} \text{ (DEFORMED BARS)}$$

$$d = d_1 + 1\tfrac{1}{2} \text{ IN.} + \tfrac{1}{2}(\text{DIAM. OF BAR}).$$

$$A_{st} = \frac{M}{15{,}000\, d_1} \text{ (PLAIN BARS) OR } \frac{M}{16{,}840\, d_1} \text{ (DEFORMED BARS)}. \quad r_o = \frac{A_{st}}{12d}$$

STRESSES. GIVEN d, d_1 AND r_o. n_1 AND a_1 AS CASE IA.

$$f_{st} = \frac{M}{12 r_o a_1 d_1 d} \qquad f_{cb} = \frac{f_{st} n_1}{15(1-n_1)}$$

RESISTANCE WHEN CRACKED.

$$M_{rs} = R_s d^2 = p_{st} r_0 \left(1 - \frac{n_1}{3}\right)\left(\frac{d_1}{d}\right) bd^2$$

in which
$$n_1 = \sqrt{\left[mr_0\left(\frac{d}{d_1}\right)\right]^2 + 2mr_0\left(\frac{d}{d_1}\right)} - mr_0\left(\frac{d}{d_1}\right).$$

When the permissible tensile stresses in the concrete (p_{ct}) and reinforcement (p_{st}) are 270 lb. and 12,000 lb. per square inch respectively, and $m = 15$, as recommended in the Code, the foregoing expressions are as given in *Table* A for a strip of slab 12 in. wide ($b = 12$ in.). The curves in the table give values of R_c and R_s for various values of $\frac{d_1}{d}$ and r_0. It is seen that for each ratio of $\frac{d_1}{d}$ there is a proportion r_{0E} of reinforcement which gives equal values of R_c and R_s, say R_E; this is the ideal case, the adoption of which gives the most economical design. If this value of R_E is substituted in $d = \sqrt{\frac{M}{R_E}}$, in which M is the bending moment to be resisted, the resulting value of d is the thickness of slab which complies with the two design requirements if the area of reinforcement A_{st} in a strip 1 ft. wide is not less than $12r_{0E}d$ sq. in.; corresponding values of r_{0E} and R_E are given in *Table* A.

Conversely, the resistances of slabs of various thicknesses with the proportion of reinforcement indicated by r_{0E} can be obtained by substitution in $M_r = R_E d^2$; the resistances and reinforcement of slabs of various thicknesses taking into account the probable value of $\frac{d_1}{d}$ are also given in *Table* A.

SLABS NOT LESS THAN 9 IN. THICK.—The foregoing basis of design applies to all slabs when the strain at the face in contact with the liquid is tensile, and to slabs less than 9 in. thick whether tensile strain occurs at the face in contact with, or remote from, the liquid. If in a slab not less than 9 in. thick, the tensile strain is at the face remote from the liquid, the method of design is as for an ordinary slab with the stress in the reinforcement not greater than 18,000 lb. per square inch in plain bars or 20,000 lb. per square inch in deformed bars, and the compressive stress in the concrete not greater than 1200 lb. per square inch. For these conditions, which apply to case IA in *Table* A, the effective depth d_1 of the slab must be not less than $\sqrt{\frac{M}{Qb}}$ in which Q is the moment-of-resistance factor and is 250 for $p_{st} = 18{,}000$ lb., and 239 for $p_{st} = 20{,}000$ lb. per square inch. The thickness of the slab must be not less than [$d_1 + 1\frac{1}{2}$ in. (cover) $+ \frac{1}{2}$ (diameter of bar)].

If the thickness of the slab provided is such that the effective depth is equal to the value calculated from $\sqrt{\frac{M}{Qb}}$, the values of A_{st} are $0{\cdot}017bd_1$, and $0{\cdot}014bd_1$ sq. in. in width b if p_{st} is 18,000 lb. and 20,000 lb. respectively. It is, however, uncommon for the effective depth to be equal exactly to the minimum depth required, and is generally greater; in this case the area A_{st} of the reinforcement in square inches in width b is given by $\frac{M}{p_{st}l_a}$ in which $l_a = 0{\cdot}83d_1$ approximately in all cases.

Conversely, the moment of resistance of a slab is given by Qbd_1^2 if the amount of reinforcement is not less than $0{\cdot}017bd_1$ (or $0{\cdot}014bd_1$ with deformed bars); the moments of resistance and corresponding amounts of reinforcement in slabs of various thicknesses are given in *Table* A for probable values of $\frac{d_1}{d}$.

Examples.—Bending Only (*Table* A).

Design walls of tanks for the conditions stated (Case IA).

(*a*).—Bending moment of 20,000 in.-lb. per foot.

This design can be taken directly from the table headed "Balanced Design", since the moment of resistance of a 6-in. slab is approximately 20,000 in.-lb. The reinforcement required is (also from the table) 0·49 sq. in. per foot, which is provided by ⅝-in. bars at 7½-in. centres with 1½-in. cover.

(*b*).—Bending moment of 24,000 in.-lb. per foot.

The curves on the table can be used directly to obtain a "balanced design." Assume $\frac{d_1}{d} = 0{\cdot}7$. (Compare with values on table headed "Balanced Design".) From curves, $R_c = R_s = R_B = 580$ and $r_e = 0{\cdot}0067$. $d = \sqrt{\frac{24{,}000}{580}} = 6{\cdot}45$ in.; that is, a 6½-in. slab is required. $A_{st} = 0{\cdot}0067 \times 6\tfrac{1}{2} \times 12 = 0{\cdot}522$ sq. in.; provide ⅝-in. bars at 7-in. centres with 1½-in. cover.

(*c*).—Check that an 8-in. slab reinforced with ⅝-in. bars at 6-in. centres with 1½-in covers and subjected to a bending moment of 36,000 in.-lb. per foot conforms to B.S. Code No. 2007.

$d_1 = 8 - 1\tfrac{1}{2} - \tfrac{5}{16} = 6{\cdot}19$ in.; $\frac{d_1}{d} = \frac{6{\cdot}19}{8} = 0{\cdot}774$; $\frac{d}{d_1} = \frac{1}{0{\cdot}774} = 1{\cdot}29$. $A_{st} = 0{\cdot}614$ sq. in.;

$r_e = \frac{0{\cdot}614}{12 \times 8} = 0{\cdot}0064$. Substitute in formulæ in *Table* A.

$$n_e = \frac{\tfrac{1}{2} + (14 \times 0{\cdot}0064 \times 0{\cdot}774)}{1 + (14 \times 0{\cdot}0064)} = 0{\cdot}523.$$

$$R_c = \frac{3240}{1 - 0{\cdot}523}\{\tfrac{1}{3} - (1 - 0{\cdot}523)\,0{\cdot}523 + [14 \times 0{\cdot}0064\,(0{\cdot}774 - 0{\cdot}523)^2]\} = 607.$$

$$d \not< \sqrt{\frac{36{,}000}{607}} = 7{\cdot}70 \text{ in., which does not exceed 8 in.}$$

$$r_s = 15 \times 0{\cdot}0064 \times 1{\cdot}29 = 0{\cdot}124;\ n_1 = \sqrt{(0{\cdot}124)^2 + (2 \times 0{\cdot}124)} - 0{\cdot}124 = 0{\cdot}39.$$

$$a_1 = 1 - (\tfrac{1}{3} \times 0{\cdot}39) = 0{\cdot}87;\ \ A_{st} \not< \frac{36{,}000}{12{,}000 \times 0{\cdot}87 \times 6{\cdot}19} = 0{\cdot}558 \text{ sq. in. per foot.}$$

which does not exceed 0·614 sq. in. Therefore design is satisfactory.

Note.—For designs in Case IB, the procedure is as in the foregoing if $d < 9$ in., and as for ordinary designs if $d \not< 9$ in.

Case II.—Slabs Subjected to Direct Tension Only.

Planar slabs subjected to direct tension only without bending are not common; a curved slab, such as the wall of a cylindrical tank, is the more common case. The two conditions of design are that the tensile stress p_{ct} in the concrete should not be so great as to cause cracking, and, should cracking occur, the reinforcement should be able to resist the whole of the tensile force T without exceeding the permissible stress p_{st}. The first of these conditions determines the thickness of the slab such that

$$T = p_{ct}[db + (m - 1)A_{st}].$$

The second condition determines the amount of reinforcement, such that $T = p_{st}A_{st}$. For the most effective design, therefore, if $A_{st} = r_e bd$, equating the two expressions for T gives

$$r_e = \frac{1}{\frac{p_{st}}{p_{ct}} - (m - 1)} \quad \text{and} \quad d = \frac{T}{[1 + (m - 1)r_e]bp_{ct}}.$$

The expressions for Case II in *Table* B give values of r_e and d for a slab 12 in. wide subjected to a tensile force T lb. per foot, if the permissible stresses p_{ct} and p_{st} are 190 lb. and 12,000 lb. per square inch respectively as recommended in the Code for 1 : 1·6 : 3·2 concrete and any type of reinforcement; $m = 15$.

Case III.—Slabs Subjected to Direct Tension Combined with Bending.

The two cases are A when the tensile strain is at the face in contact with the liquid, and B when this face is subjected to compressive strain and the opposite face to tensile strain.

CASE IIIA. TENSILE STRAIN AT FACE IN CONTACT WITH LIQUID.—There are two conditions for this case, namely, when tensile stresses only are produced, and when tensile and compressive stresses are produced. If the direct tension is T and the bending moment is M, $e = \frac{M}{T}$.

(a) *Tensile Stresses Only.*—This condition, which occurs when e does not exceed $d_1 - \frac{1}{2}d$, is analogous to a slab subjected to direct tension only and therefore the tensile stress in the concrete must not exceed the stress p_{ct} permissible in direct tension if no cracking is to occur. Should cracking occur, the permissible tensile stress p_{st} in the reinforcement must not be exceeded. The latter condition determines the amount of reinforcement and if there is the same amount of reinforcement near each face, the proportion r_0 of reinforcement near each face must be not less than given by

$$r_0 = \frac{T}{2p_{st}bd}\left[1 + \frac{e}{d_1 - \frac{1}{2}d}\right].$$

The permissible tensile stress in the concrete is not exceeded if

$$\frac{T}{bd}\left[\frac{1}{1 + 2r_0(m-1)} + \frac{e}{\left[\frac{1}{6} + 4r_0(m-1)\left(\frac{d_1}{d} - \frac{1}{2}\right)^2\right]d}\right] \ngtr p_{ct}.$$

The thickness of the slab is determined from this requirement.

Evaluation of these expressions for $p_{ct} = 190$ lb. per square inch, $p_{st} = 12{,}000$ lb. per square inch, $b = 12$ in., and $m = 15$, are given in *Table* B.

(b) *Tensile and Compressive Stresses.*—This condition, which occurs when e exceeds $d_1 - \frac{1}{2}d$, is analogous to a slab subjected to bending only, and therefore the limiting stresses for that condition apply. Since the effect of reinforcement in the compression zone of a slab is insignificant, such reinforcement, if any, can be neglected. Given dimensions d and d_1, the proportion r_0 of tensile reinforcement required to ensure that the steel is not overstressed should cracking occur is given by

$$r_0 = \frac{T}{p_{st}bd}\left(1 + \frac{e + \frac{1}{2}d - d_1}{l_a}\right).$$

The position of the neutral plane if cracking does not occur is given by

$$n_0 = \frac{\frac{1}{2} + (m-1)\left(\frac{d_1}{d}\right)r_0}{1 + (m-1)r_0}.$$

To ensure cracking does not occur, the slab must be thick enough, and this is so if

$$\frac{T}{bd}\left[\frac{1}{1 + (m-1)r_0} + \frac{e(1 - n_0)}{d\left[\frac{1}{3} - (1 - n_0)n_0 + r_0(m-1)\left(\frac{d_1}{d} - n_0\right)^2\right]}\right] \ngtr p_{ct}.$$

The corresponding expressions in *Table* B are for the condition $p_{ct} = 270$ lb. per square inch, $p_{st} = 12{,}000$ lb. per square inch, $b = 12$ in., and $m = 15$.

Table B.—Direct Force with or without Bending.

IN ACCORDANCE WITH B. S. CODE No. 2007.

CONCRETE: NOMINAL PROPORTIONS 1 : 1·6 : 3·2 $m = 15$

MINIMUM PROPORTION OF REINFORCEMENT: $r_0 = \frac{A_{st}}{bd} \nless 0.003$.

MINIMUM COVER: 1½ IN.

DIRECT TENSILE FORCE = T LB. PER FOOT. BENDING MOMENT = M IN.-LB. PER FT.

CASE II — CONCENTRIC TENSILE FORCE ONLY

DESIGN FOR STRENGTH.

DETERMINES REINFORCEMENT A_{st}. $p_{st} \ngtr 12{,}000$ LB./SQ.IN.

$$A_{st} = \frac{T}{12{,}000} \text{ SQ.IN./FT. OR } r_0 = \frac{T}{144{,}000d}.$$

RESISTANCE TO CRACKING

DETERMINES THICKNESS d: $p_{ct} \ngtr 190$ LB./SQ.IN.

$$d \nless \left[\frac{T}{2280} - 1.17A_{st}\right] \text{ IN.}$$

BALANCED DESIGN.

$$d = \frac{T}{2930} \text{ IN.} \qquad A_{st} = 0.244d \text{ SQ.IN./FT.}$$

CASE IIIA — TENSILE FORCE COMBINED WITH BENDING (TENSILE STRESS AT FACE IN CONTACT WITH LIQUID)

$$e = \frac{M}{T}$$

(a) $e \ngtr (d_1 - \frac{1}{2}d)$. TENSILE STRESSES ONLY. $p_{ct} \ngtr 190$ LB./SQ.IN. $p_{st} \ngtr 12{,}000$ " "

GIVEN OR ASSUME d AND d_1

DETERMINE $r_0' = \frac{T}{288{,}000d}\left[1 + \frac{e}{d_1 - \frac{1}{2}d}\right]$ [PROVIDE A'_{st} AT BOTH SIDES]

SUBSTITUTE IN $f_{ct} = \frac{T}{12d}\left[\frac{1}{1+28r_0'} + \frac{ed}{\frac{1}{6}d^2 + 56r_0'(d_1 - \frac{1}{2}d)^2}\right] \ngtr 190$

ADJUST d UNTIL EXPRESSION IS SATISFIED.

(b) $e > (d_1 - \frac{1}{2}d)$. TENSILE AND COMPRESSIVE STRESSES. $p_{ct} \ngtr 270$ LB./SQ.IN. $p_{st} \ngtr 12{,}000$ " "

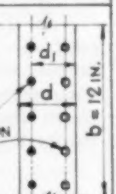
GIVEN OR ASSUME d AND d_1

DETERMINE $r_0 = \frac{T}{144{,}000d}\left[1 + \frac{e + \frac{1}{2}d - d_1}{0.83d_1}\right]$

$$n_0 = \frac{\frac{1}{2} + 14r_0\frac{d_1}{d}}{1 + 14r_0}$$

SUBSTITUTE IN $\frac{T}{12d}\left\{\frac{1}{1+14r_0} + \frac{e(1-n_0)}{\left[\frac{1}{3} - (1-n_0)n_0 + 14r_0\left(\frac{d_1}{d} - n_0\right)^2\right]d}\right\} \ngtr 270$

ADJUST d UNTIL EXPRESSION IS SATISFIED.



CASE IIIB — TENSILE FORCE COMBINED WITH BENDING (TENSILE STRESS AT FACE REMOTE FROM LIQUID)

(a) $d < 9$ IN. ADOPT CASE IIIA.

(b) $d \nless 9$ IN. $p_{st} = 18{,}000$ LB./SQ.IN. (20,000 LB./SQ.IN. FOR DEFORMED BARS) ($p_{cb} \ngtr 1200$ LB./SQ.IN. – NOT CRITICAL). DIAGRAM AS CASE IIIA (b)

GIVEN OR ASSUME d AND d_1 $\left[d_1 \nless \sqrt{\frac{M}{3000}} \text{ (PLAIN BARS) OR } \sqrt{\frac{M}{2872}} \text{ (DEFORMED BARS)}\right]$

$$e_s = e + \tfrac{1}{2}d - d_1$$

$$A_{st} = \frac{T}{18{,}000 \text{ OR } 20{,}000}\left[1 + \frac{e_s}{0.83d_1}\right] \text{ SQ.IN./FT.}$$

IF d, d_1 AND A_{st} GIVEN :- $f_{st} = \frac{T}{A_{st}}\left[1 + \frac{e_s}{0.83d_1}\right] \ngtr 18{,}000$ (OR 20,000)

CASE IIIB. TENSILE STRAIN AT FACE REMOTE FROM LIQUID.—Two conditions are to be considered, namely, when the slab is less than and not less than 9 in. thick.

(a) *Slab less than 9 in. thick.*—For this case the design should be exactly as for Case IIIA, applying sub-case (a) or (b) as applicable.

(b) *Slab not less than 9 in. thick.*—If d and d_1 are known, or assumed, the area of tensile reinforcement required is calculated from

$$A_{st} = \frac{T}{p_{st}}\left(1 + \frac{e_s}{l_a}\right)$$

in which $e_s = e + \frac{1}{2}d - d_1$.

An approximate value for d_1 (from which d can be determined) is $\sqrt{\frac{M}{Qb}}$ in which Q is the moment of resistance factor corresponding to the permissible stresses p_{st} and p_{cb} in the steel and concrete respectively. If M is large in respect to T, that is, if e is large, say greater than d_1, the value of d_1 required to ensure that p_{cb} is not exceeded should differ much from the value calculated from the foregoing expression; if e is smaller than d_1 a slightly thinner slab may be sufficient. In doubtful cases, the maximum compressive stress should be calculated by one of the ordinary methods of combined stress.

The expressions in subsection (b) of Case IIIB in *Table* B apply to the conditions p_{st} = 18,000 lb. for plain bars or 20,000 lb. per square inch for deformed bars, p_{cb} not exceeding 1200 lb. per square inch, b = 12 in., and m = 15.

Examples.—Tension with and without Bending (*Table* B).

Design walls of tanks for the conditions stated.

(*a*).—Direct tensile force of 25,000 lb. per foot (no bending).—Case II.

(i) $d = \frac{25{,}000}{2930} = 8{\cdot}53$ in.; say $8\frac{1}{2}$-in. slab.

$A_{st} = 0{\cdot}244 \times 8{\cdot}5 = 2{\cdot}07$ sq. in. per foot; provide 1-in. bars at 9-in. centres in each of two rows.

(ii) Alternative method. $A_{st} = \frac{25{,}000}{12{,}000} = 2{\cdot}08$ sq. in. per foot; provide 1-in. bars at 9-in. centres (total = 2·09 sq. in.) in each of two rows.

$$d \not< \left[\frac{25{,}000}{2280} - (1{\cdot}17 \times 2{\cdot}09)\right] = 8{\cdot}53 \text{ in.; that is, } 8\tfrac{1}{2}\text{-in. slab.}$$

(*b*). Tensile force of 12,000 lb. per foot, and bending moment of 20,000 in.-lb. per foot. Assume d = 8 in. and $d_1 = 8 - 2 = 6$ in.; $d_1 - \frac{1}{2}d = 6 - 4 = 2$ in.

$$e = \frac{20{,}000}{12{,}000} = 1{\cdot}67 \text{ in. } (\not> 2 \text{ in.; therefore Case IIIA } a \text{ applies.})$$

$$r_e' = \frac{12{,}000}{288{,}000 \times 8}\left(1 + \frac{1{\cdot}67}{2}\right) = 0{\cdot}0095.$$

$$f_{ct} = \frac{12{,}000}{12 \times 8}\left[\frac{1}{1 + (28 \times 0{\cdot}0095)} + \frac{1{\cdot}67 \times 8}{(\frac{1}{6} \times 8^2) + (56 \times 0{\cdot}0095 \times 2^2)}\right]$$

= 230 lb. per square inch (approx.) which exceeds 190 lb. per square inch. Recalculate with d = 9 in., and d_1 = 7 in.; $d_1 - \frac{1}{2}d = 2\frac{1}{2}$ in.

$r_e' = 0{\cdot}0077$; f_{ct} = 195 lb. per square inch ($\simeq$ 190); satisfactory if A_{st} is slightly greater than $12 \times 0{\cdot}0077 \times 9 = 0{\cdot}83$ sq. in. per foot; provide say 1-in. bars at $10\frac{1}{2}$-in. centres at each face (total = 0·898 sq. in. per foot).

(*c*).—Tensile force of 10,000 lb. per foot and bending moment of 48,000 in.-lb. per foot. Assume d = 10 in. and d_1 = 8 in.; $\frac{d_1}{d} = 0{\cdot}8$; $e = \frac{48{,}000}{10{,}000} = 4{\cdot}8$ in. ($> 8 - 5 = 3$ in.; therefore Case IIIA b applies.)

$$r_e = \frac{10{,}000}{144{,}000 \times 10}\left(1 + \frac{4{\cdot}8 + 5 - 8}{0{\cdot}83 \times 8}\right) = 0{\cdot}0088; \quad 14r_e = 0{\cdot}122.$$

$$n_e = \frac{0{\cdot}5 + (0{\cdot}122 \times 0{\cdot}8)}{1{\cdot}122} = 0{\cdot}532.$$

$$f_{ct} = \frac{10{,}000}{12 \times 10}\left\{\frac{1}{1{\cdot}122} + \frac{4{\cdot}8(1 - 0{\cdot}532)}{[\frac{1}{3} - (1 - 0{\cdot}532)0{\cdot}532 + 0{\cdot}122(0{\cdot}8 - 0{\cdot}532)^2]10}\right\}$$

$= 275$ lb. per square inch ($\triangleq 270$); satisfactory if A_{st} is slightly greater than $12 \times 0{\cdot}0088 \times 10 = 1{\cdot}06$ sq. in. per foot; provide 1-in. bars at 8-in. centres (1·178 sq. in.) at tensile face.

Note.—For designs in Case IIIB, the procedure is as in the foregoing if $d < 9$ in., and as for ordinary designs if $d \nless 9$ in.

Gap-graded Aggregate in Concrete for Marine Works.

WHAT is probably the first instance in this country of gap-graded aggregate being used in concrete for marine works is in the reconstruction of the sea-wall at Dymchurch, Kent. The wall, which is to be improved and strengthened, is along a very exposed coast-line, and is nearly a mile in length, extending between the Grand Redoubt and Willop Sluice. Separate tenders were invited for reconstruction of the wall in granite, or in ordinary dense concrete, or in any other form of construction proposed by the contractors tendering. The cost of construction in granite was excessive. The tender accepted is for precast concrete blocks and cast-insitu concrete using gap-graded aggregate. In preliminary tests the resistances to abrasion, frost and absorption of this type of concrete were examined. The results indicate that concrete of suitable mixture with gap-graded aggregate might well prove superior to concrete with continuously-graded aggregate especially as regards resistance to abrasion.

The profile of the existing apron is to be re-formed by means of plain concrete filling and the sea-wall is to be extended about 19 ft. seawards to a new row of timber sheet-piles. The wider apron, the increased total width of which will be about 80 ft., will have the effect of reducing the action of waves on the wall. The precast blocks, which will be made in a casting yard established on the site, will form the surface of the new apron. Towards the upper part of the apron a concrete berm 20 ft. wide will be constructed. On the top of the wall a roadway 15 ft. wide will be constructed with a parapet on the landward-side as a protection against waves washing over and scouring out the ground behind the wall. Various ramps and steps will also be constructed.

There is no record of the time when the first clay sea-wall was built at Dymchurch but it is known to have undergone several alterations during its lifetime. In recent years it became apparent that repairs were needed along the section of the sea-defence works where reconstruction is now to be carried out. Abrasion has worn away much of the stone facing of the old apron, and from time to time damage by storms to the middle section has necessitated costly repairs. The large quantities of water carried over by onshore gales have on occasions scoured out parts of the back of the wall. To reduce the risk of further damage, the action of waves on the wall will be mitigated by creating shallower water at the middle of the apron and by laying heavy concrete blocks.

The work is being carried out for the Kent River Board under the direction of the Chief Engineer, Mr. J. I. Taylor, M.B.E., M.I.C.E. The probable cost is £430,000 and the contract time is three years, but it is hoped that the work will be completed in less time. The contractors are Messrs. Holland & Hannen and Cubitts (Great Britain) Ltd. The consulting engineers for the development of the gap-graded concrete are Messrs. Sandberg.

A Newspaper Office in London.

THE new office and printing press building (*Fig.* 1) for the "Daily Mirror" has a maximum height of 169 ft. above the street and is of reinforced concrete construction throughout. In the main building there are twelve storeys above ground, with mezzanine floors in the ground floor and top storeys, and four basement floors where the printing presses, paper stock, and foundry are located. The basement covers the entire area of the site, and the ground and first floors cover most of this area. Above the third floor a tower block covers only part of the site. The primary factor in the design of the building was the installation of the printing presses, each of which weighs about 820 tons. Two presses are already installed, but eventually there will be five, which will cover a floor area of 24,000 sq. ft. The building is shown in the course of construction in *Fig.* 2.

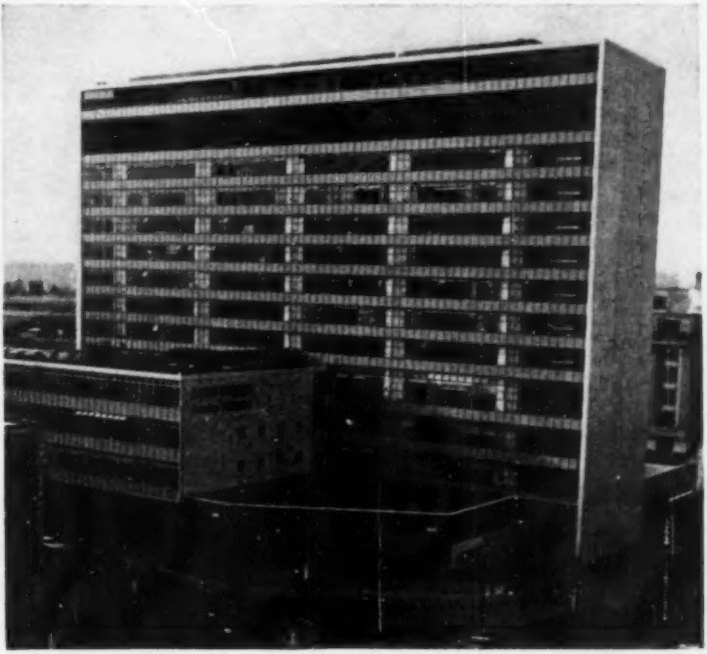


Fig. 1.

Basements.

Work was begun on the site in 1956. Excavation for the basement retaining walls was carried out in trenches 15 ft. wide strutted from a central dumpling of earth. Excavation extended to 45 ft. below the street level for the lowest basement floor, and to over 60 ft. for the lift-shafts. This method of construction permitted the central area of the site to be used for the storage of plant and materials while excavation for the retaining walls proceeded around the perimeter of the site.

The retaining walls are designed to span vertically between the base of the wall and the ground floor, and to withstand horizontal pressure of 30 lb. per square foot per foot of depth. The calculated pressure on the ground at the base of the wall is about 3 tons per square foot. The

subsoil over the entire site is London blue clay. The trench was excavated by hand with pneumatic clay-spades. The walls were built in 4-ft. lifts. The vertical face of the clay was lined with precast concrete planks 3 in. thick and 4 ft. in length. The planks were then covered with asphalt and the concrete for the walls was cast against them. For lengths of 15 ft. on the centre-lines of the columns, the walls are 8 ft. thick, and 5 ft. 6 in. thick for the intervening lengths of 30 ft. On the south side, the wall is 5 ft. thick with an additional 2 ft. of plain concrete to carry

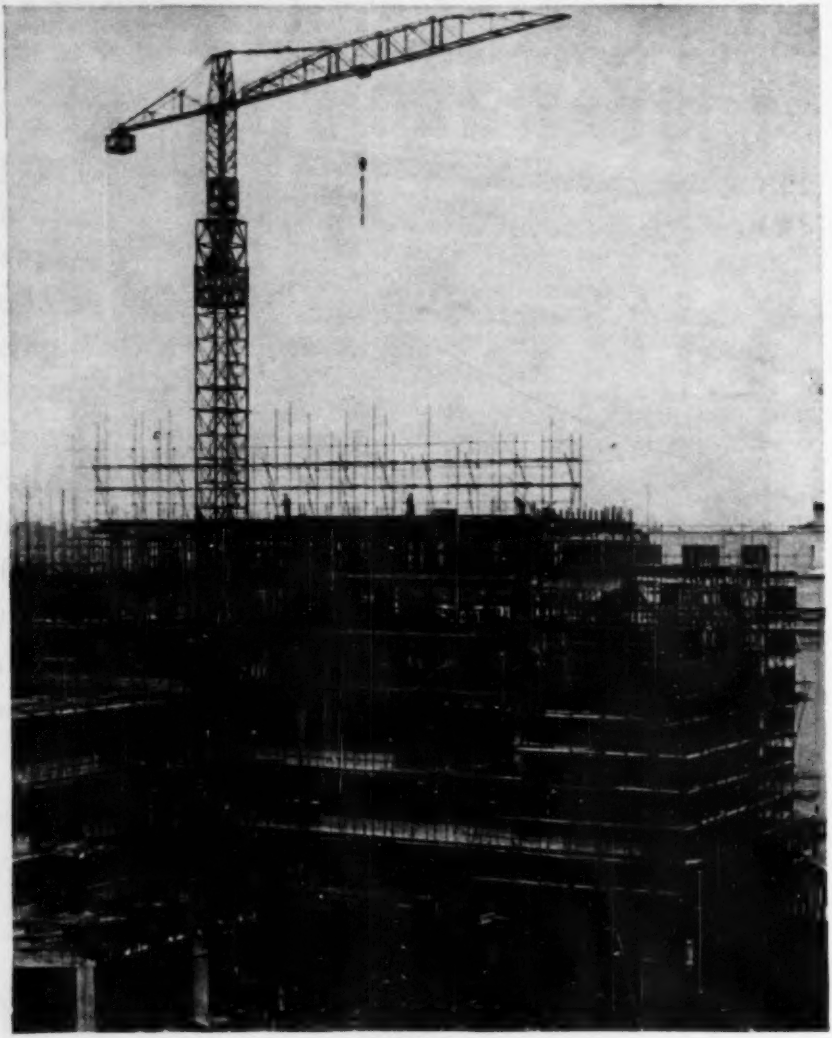


Fig. 2.

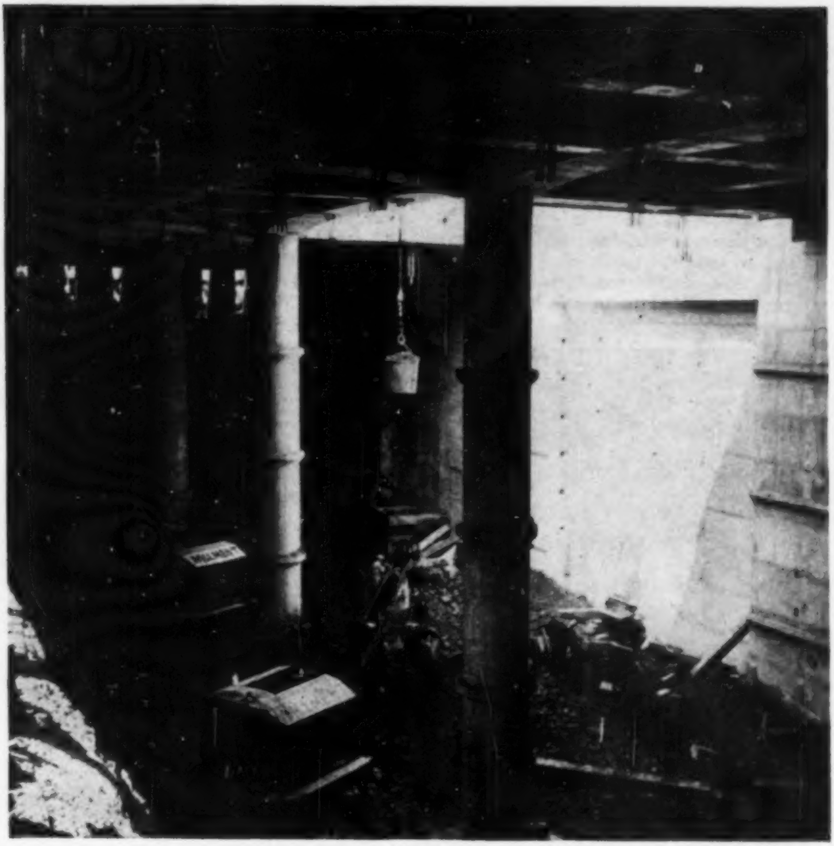


Fig. 3.

the foundations of adjacent existing buildings.

The twenty main columns (*Fig*. 3) in the basements are at 45-ft. centres in both directions and are arranged in five rows of four columns each. At first an excavation 15 ft. square was made for each column base, which in its initial state was sufficient to support the weight of the construction up to and including the second floor. The greatest load on one of the main columns in the completed building is about 3000 tons. The columns are 4 ft. 6 in. in diameter at the bottom and are reinforced with 112 1⅜-in. bars and 1-in. diameter helical binding at 3-in. centres.

The main columns have projecting corbels (*Fig*. 3) supporting the three intermediate basement floors, which are flat slabs carried on secondary concrete columns 12 in. square. The upper basement floor is hung from the ground-floor beams by hangers at 15-ft. centres, so that the second basement is completely free of intermediate columns as it is required for the storage of reels of newsprint. Because of the restricted site, vehicles bringing aggregate and other materials and those removing the excavated spoil had to travel over the ground floor, but since the ground floor is designed to carry an internal roadway and the floor of the first basement, it

was sufficiently strong to carry this temporary vehicular load.

After the first 80-ft. length of the ground floor from the northern frontage was completed, which amount was calculated to be sufficient to withstand the horizontal thrust from the top of the retaining wall, the excavation of the dumpling, containing 55,000 cu. yd. of clay, was commenced to enable the construction of the basement raft foundation.

The raft was combined with the column bases so that the entire final load on the columns is distributed more or less uniformly over the entire area of the basement. The final pressure on the ground under the raft is about 2 tons per square foot which is equivalent to the weight of the material excavated. The whole raft, in addition to being made watertight by a layer of asphalt, overlies porous concrete containing agricultural land-drains connected to stand-pipes embedded in the retaining walls, by which means any water pressure under the raft is relieved. Concreting began with the construction of that part of the raft adjacent to and underpinning the primary column bases and then spread out to the remaining areas.

Superstructure.

The main beams of the first and second floors are 4 ft. wide and span between the main columns, but on the central row of columns the beams are divided into two 2-ft. ribs with a 4-ft. gap between them. This arrangement allows vertical ducts and services to run uninterruptedly throughout the height of the building. Secondary beams at 15-ft. centres are provided resulting in a grid of beams at 15-ft. centres in two directions.

In the tower, above the third floor, the beams are at 15-ft. centres in a north–south direction only. The columns remain at 45-ft. centres but are rectangular in cross-section except at the northern and southern extremities. Three arched frames (*Fig.* 4) of 45-ft. span are provided at the third floor to carry the columns supporting the floors in the tower. The beams at the foot of each arch include additional reinforcement to resist a direct tension of 500 tons induced by the arches; therefore each beam 2 ft. wide contains fifty-six 1⅛-in. bars.

Fig. 4.

Holes 4½ in. or 9 in. in diameter are formed at 3-ft. centres in the ribs of all beams to enable pipes and other services to pass through without the need for cranking under the beams.

The heating, ventilating and compressed-air plant and the lift-motor rooms are in the top storey of the tower. The height of this storey is 30 ft., this height being required for the lift-motor rooms. An intermediate floor is provided to accommodate the plant and is suspended from the roof beams, thereby reducing the load on the eleventh floor. The walls

of the room housing the plant are of light-weight insulating concrete blocks 3 in. thick. The reinforced concrete slab of the eleventh floor is 5 in. thick; on this there is a floating floor comprising a concrete slab, about 4 in. thick, carried on 1½-in. by 1½-in. cork strips forming squares 10 in. by 10 in. The gap between the squares is filled with cork granules and this insulating layer is covered with ½ in. of corrugated paper and roofing felt.

The lifts are at the north and south of the building. The concrete walls of the lift-shafts were constructed independently of the structural floors and were subsequently tied to the floors by means of bars projecting from the floors. Similarly, bars were left projecting to connect to the stairs constructed subsequently.

Loadings.

The floors in the tower are designed for the ordinary imposed office loading of 60 lb. per square foot. At the northern and southern ends of the floors in the tower, the circular columns are retained by beams 4 ft. wide spanning between them and acting as wind bracing; they are designed to withstand the wind pressure on the flank walls of the tower.

The production floors and the main editorial floor are designed for an imposed load of 200 lb. per square foot. The ground floor is designed for an imposed load of 500 lb. per square foot.

To obtain a sufficient pressure, water is stored at two levels. On the eleventh floor, two reinforced concrete tanks having a combined capacity of 34,500 gallons are provided to supply all the floors in the tower block. On the third floor, five reinforced concrete tanks having a total capacity of 34,000 gallons supply all the floors below this level, and are suspended from the arches shown in *Fig.* 4.

External Finishes.

The walls of the ground-floor storey are faced with riven Westmorland dark green slate slabs, which provide a rough-textured base to the building. The northern flank walls are faced with white marble. The northern and southern end walls of the tower are faced with artificial stone with Derbyshire-spar aggregate.

The joint architects and engineers are Sir Owen Williams & Partners and Messrs. Anderson, Forster & Wilcox. The contractors were Messrs. W. & C. French Ltd.

Engineers: Qualifications and Earnings.

THE following has been received from The Engineers' Guild.

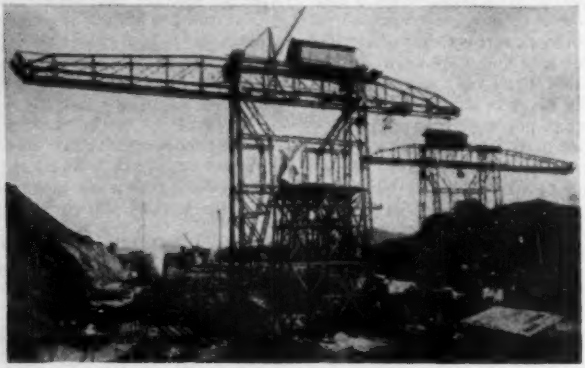
"Your very interesting editorial in the April number discusses the booklet entitled 'Professional Engineers' in the 'Choice of Career' series and also the results of the recent survey of engineers' salaries conducted by the Engineers' Guild. The former, as you point out, states that much of the reward of following the profession is the satisfaction to be obtained from mastering problems and from hard and creative work and your apt comment on this is 'It is well that an engineer may have these satisfactions because material rewards may not be so generous.'

The Guild's survey of salaries shows that the remuneration of professional engineers has considerably increased over the last five years but there is, at present, no evidence that their position relative to other comparable employments has improved. Something more would appear to be required at this stage to attract sufficient numbers of young men to the profession. In the absence of a still better financial return for their work, the extra inducement might be provided by greater prestige and higher status or preferably by both together. You did not deal with this latter aspect, but the Engineers' Guild thinks it very important. Many men (amongst whom may be numbered cabinet ministers) who could obtain better salaries elsewhere, take posts offering less financial remuneration because of the status, prestige and satisfaction attached to such posts. By the use of similar incentives, more engineers would be attracted to the profession and this in turn would provide the enhancement to their status and prestige so long overdue."

FIFTY YEARS AGO.

From "Concrete and Constructional Engineering", June, 1911.

Concrete Construction in the Panama Canal.



"The plant used for placing concrete in the two flights of locks on the Pacific slope consists primarily of eight cantilever cranes (as illustrated) of two types, designated respectively, berm and chamber cranes. The berm cranes are equipped with double fixed cantilevers, 150 ft. long, and are 62 ft. 6 in. high to the bottom of the cantilever, and 90 ft. high to the top of the machinery house. The towers are supported by two box girders, each carried on 16 axles. There are two trolleys to each crane, each carrying a 2½ cu. yd. bucket clamshell, which supplies sand and crushed rock from the storage piles beneath the outer ends of the cantilever arms. Each trolley has a rated capacity per hour of 50 cu. yd. of sand and 100 cu. yd. of crushed rock. In each tower are four bins—two for sand and two for crushed rock, each of 15 cu. yd. capacity. Beneath the bins is a platform, where the bags of cement are opened and emptied into hoppers. The cement is transferred to the platform at the rate of 800 bags an hour. The measuring hoppers, which supply the concrete mixers, are placed directly below the cement platform. Each contains three compartments—for sand, crushed rock, and cement respectively. In the tower bases are two concrete mixers, each of 2 cu. yd. capacity, which discharge into 2 cu. yd. buckets carried on flat cars. The operators on the mixer platforms control the discharges by means of compressed air, and also work the chute doors which admit the raw material into the mixers from the measuring hoppers. Water is fed into the mixers from automatic measuring tanks installed in the towers; an instrument records the time consumed in each mixing operation.

The chamber cranes were mainly designed to place the concrete in the lock walls; but are also used in transferring the wood and steel forms from one point to another. Each crane is 97 ft. 6 in. high to the bottom of the cantilever, and 115 ft. high to the top of the machinery house. There are two cantilever arms to each tower—one extended towards the centre wall with a length of 53 ft. 6 in., and the other, 81 ft. 6 in. long, passing over the side wall. The buckets handled by these cranes are of the double-leaf drop-bottom type, and have a capacity of 2 cu. yd. each. The cranes pick up the buckets, deliver them to the lock wall, dump their contents, and deliver the empties to the cars at an average rate of 320 cu. yd. of concrete an hour.

At points in the centre and side walls thermometers were embedded as the concrete reached the successive heights. A complete graphical record of the hardening and cooling of the concrete will be obtained."

Bulletins Received.

"**Structural Frameworks for Single-storey Factory Buildings.**" By H. V. Apcar. Department of Scientific and Industrial Research. Factory Building Studies No. 7. (London: H.M.S.O. 1960. Price 4s.)

This paper describes how costs are affected by type of roof, span of roof, column spacing, clear height, and loading. A general discussion of the influence of structural type on the satisfaction of other functional requirements is given.

"**Experiments on Concrete Bars: Expansions during Storage in Climate Room.**" By A. Efsen and O. Glarbo. (Copenhagen: The Danish National Institute of Building Research. 1960. In English. Price Kr. 12.)

This document is an interim report of research now proceeding under the auspices of the Committee on Alkali Reactions in Concrete.

"**Beiträge zur Lösung von Scheibenproblemen.**" By R. Mathys. (Zürich: Verlag Leeman. In German. 1960. Price Swiss Fr. 12.)

"**Platten mit freien Rändern.**" By H. von Gunten. (Zürich: Verlag Leemann. In German. 1960. Price Swiss Fr. 12.)

"**Archiwum Inzynierii Ladowej.**" (Warsaw: Polska Akademia Nauk. Vol. VI. In Polish. 1960. Price 25 Zlote.)

"**Thermal Conditions in the Ground from the Viewpoint of Foundation Work, Heating and Plumbing Installations and Draining.**" By O. Vuorelainen. (Helsinki: State Institute for Technical Research, Finland. In English. 1960. No price stated.)

"**Bulletin of the Polytechnic Institute of Jassy.**" (Jassy: In Roumanian, Russian, French and English. New Series Vol. V. 1959. No price stated.)

"**The Problem of the Safety of Arch Dams.**" By M. Rocha and J. Laginha Serafim. Memorandum No. 142. National Laboratory of Civil Engineering of Portugal. (In Portuguese. 1960. No price stated.)

Deals with studies of models.

"**R.I.L.E.M. Bulletin.**" Nos. 8 and 9. (In French and English. 1960. No price stated.)

Bulletin No. 8 contains the papers on "Model Structures" presented at the Symposium in Madrid in 1959. Bulletin No. 9 contains the papers on the "Influence of Time upon Strength and Deformation of Concrete" presented at the Symposium in Munich in 1958.

A Large Prestressed Concrete Transmission-line Tower.

WHAT is thought to be the largest prestressed concrete transmission-line tower in the world has been erected near Pittsfield, Massachusetts, U.S.A. The structure is 99 ft. high and weighs 74 tons. The two legs of the tower are embedded for a length of 4 ft. 6 in. into sockets in twin reinforced concrete bases, each of

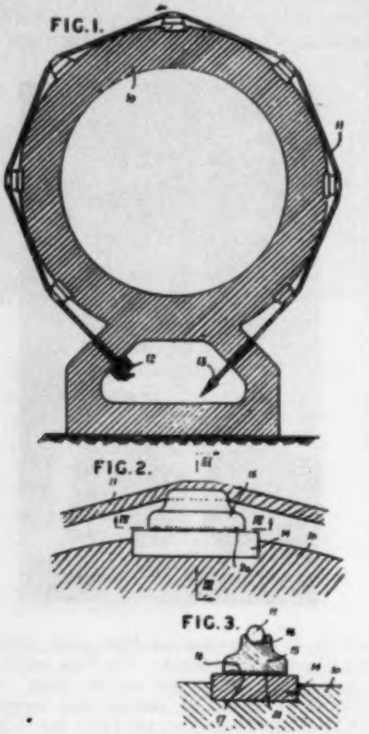
which is supported on four piles driven 80 ft. into the ground. The legs and the cross-arms, which are 93 ft. long, are hollow in order to reduce the weight. The legs and cross-arm are 18 in. by 36 in., the thickness of the wall being 5 in. The braces comprise four solid members 12-in. square, pin-jointed at the columns, and welded and grouted at the central meeting point. The tower is stabilised by wire guys.

The tower was supplied by the Prestressed Concrete Institute (of U.S.A.) as a contribution to a project for which the General Electric Co. (of U.S.A.) is responsible. The accompanying illustration is from "Concrete Products", February 1961.

Patent Applications for Prestressed Concrete.

Culverts and Pipes.

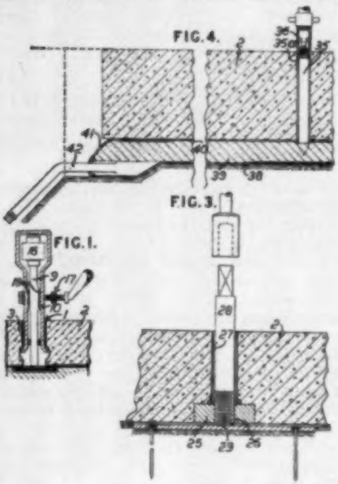
Apparatus for prestressing constructions 10 of concrete by means of a tensioned cable 11 comprises movable shoes 15, having grooves 16 in which the cable 11 is located, which are adapted to slide on



fixed shoes 14 embedded in the concrete, lubricant being retained in recesses 18 in the bases of the shoes 15 by packing 20. The shoes are made of cast iron and the bearing surfaces are treated with sulphur. A plurality of recesses may be formed in the base of each shoe 15. The lubricant is preferably a mixture of bisulphide of molybdenum and thick grease with a small quantity of colloidal graphic. Conduits may be provided through which lubricant under pressure is supplied.—No. 840,982. Ateliers Partiot Cementation. April 26, 1957.

Runways and Roads.

In the prestressing of concrete slab structures such as airfield runways, roads or the like, each concrete slab is lifted, as by jacking means, so that it is supported clear of the ground and a prestress is applied to the slab after or during lifting of the slab. The jacking means may be applied to lift and support each slab by forming a plurality of holes in spaced positions in each slab through which jacking thrust bars can project down to the surface of the ground beneath the slab or



preferably on to a bearing plate on the ground. The jacking means may comprise a block 25, *Fig.* 3, secured on the underside of the slab 2 when cast, and having a threaded bore 26 adapted to receive a screw threaded lower end 29 of a rotatable thrust bar 28. The block 25 is located at the lower end of a steel lining tube 27. Alternatively hydraulic or pneumatic jacking means 16 may be applied between the upper end of the thrust bar 9, *Fig.* 1, and a stirrup 10 which is wedged into a steel tube 3 lining the hole 1 in the slab 2. After the jacking operation is completed the stirrup 10 and the bar 9 may be locked together by means of a clamp 17 and opposed serrated surfaces 18, so as to enable the jacking means 16 to be removed whilst maintaining the support of the slab at this point. In an

alternative embodiment, each slab is lifted by water injected under pressure underneath the slab through non-return valves 36, *Fig.* 4, and tubes 35*a* lining holes 35 in the slab. In order to prevent escape of water from beneath the slab a thin waterproof membrane 38, e.g. of polyethylene sheet, is provided. A layer of coarse sand 39 is laid over the membrane 38 and is covered by a layer of building paper 40. The membrane 38 is turned back inwardly at its margins 41 to overlie the margins of the building paper 40. A number of drainpipes 42 are provided for draining water from below the slab after the lifting operation.—No. 853,418. D. H. Lee. March 24, 1959.

Courses at the Road Research Laboratory.

COURSES will be held at the Road Research Laboratory during the autumn of 1961 and, as in previous years, they will deal with the fundamental properties of road materials and with the results of research and their application in modern practice. The subjects and date of some of the courses are as follows.

Concrete.—26 September to 5 October and 10 to 19 October.

Soil Mechanics.—21 to 28 November, and 5 to 12 December.

The fee for each course is £12 12*s*. Applications must be received by the 29 June, 1961. Forms of application can be obtained from the Director, Road Research Laboratory, Harmondsworth, West Drayton, Middlesex.

SITUATION VACANT. Designer of shuttering and builders' plant required by manufacturer. Concrete mould experience advantageous. Full details of experience and qualifications to The Chief Engineer, A.B. Mould & Construction Co. Ltd., Vulcan Works, New Addington, Croydon.

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SITUATIONS VACANT. Clarke, Nicholls & Marcel require experienced reinforced concrete designer-detailers and draughtsmen for their Bristol and Cardiff offices. Positions are pensionable, and offer first-class experience for those studying for professional qualifications. A senior structural engineer is also required for the Cardiff office, and preference would be given to an A.M.I.Struct.E. Apply to 10 Apsley Road, Bristol 8, or South Wales Institute of Engineers Building, Park Place, Cardiff.

SITUATIONS VACANT. Consulting engineer, W. Paszkowski, D.I., A.M.I.C.E., has vacancies for assistant engineers and reinforced concrete detailers. Variety of civil and structural work. Apply, 1 Gregory Place, Kensington, London, W.8.

SITUATION VACANT. Girlingstone require a qualified reinforced concrete engineer for insitu and precast construction. The appointment has excellent opportunities and pension scheme, and headquarters could be at Leeds or Feltham, Middx. Apply by letter only, with details of experience, salary, age, etc., to Girlingstone, Feltham, Middx.

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(Mr. R. H. Walpole—General Industrial Design)
or write to the

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Southall, Middlesex.

Alternatively, senior personnel will be available for informal discussions about these posts at our offices at 10 Park Street, London, W.1, from 5.30 p.m. on the Thursday following this advertisement.

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Applicants should be of H.N.C. or graduate standard with 7 to 10 years' experience and capable of supervising small drawing office engaged on reinforced concrete and structural steelwork. Salary £900 to £1,200 according to experience. Write, giving age, qualifications and experience, to Box 4763, Concrete and Constructional Engineering, 14 Dartmouth Street, London, S.W.1.

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SITUATION WANTED.

SITUATION WANTED. Reinforced concrete designer-detailer, 27, seeks job, not necessarily London area, where design office and site experience are available side by side. Willing to enter into a contract on reasonable conditions. Box 4761, CONCRETE AND CONSTRUCTIONAL ENGINEERING, 14 Dartmouth Street, London, S.W.1.

BUSINESS OPPORTUNITY.

BUSINESS OPPORTUNITY. Consulting structural engineer with office space and drafting capacity available in Victoria Street in excess of present requirements and having continuity of work from small number of clients would welcome suggestions for association with established or aspiring consultants or others. Please reply, in confidence, to Box 4764, CONCRETE AND CONSTRUCTIONAL ENGINEERING, 14 Dartmouth Street, London, S.W.1

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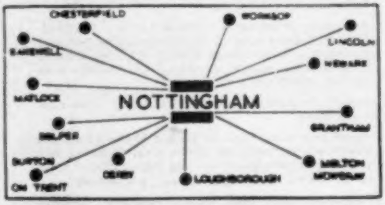
PATENTS.

PATENT. The proprietor of British Patent No. 771,397, which relates to a Concrete Block Forming Machine, is desirous of exploiting the invention by way of licence or sale. Enquiries should be addressed to ANDREWS & BYRNE, 201 Bank Chambers, 329 High Holborn, London, W.C.1.



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